PB2002-106765

COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION

PENNDOT RESEARCH

HEAVY AXLE STUDY: IMPACT OF HIGHER RAIL CAR WEIGHT LIMITS ON SHORT-LINE RAILROADS

University-Based Research, Education, and Technology Transfer Program

AGREEMENT NO. 359704, WORK ORDER 27

VOLUME I: FINAL REPORT

January 2002

By J. A. Laman, C. Leighty, G. L. Gittings, E. Erdogmus and T. E. Boothby

PENNSTATE

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Technical Report Documentation Page

<table>
<tr><td colspan="2">1. Report No.
FHWA-PA-2001-011-97-04 (I)</td><td>2. Government Accession No.</td><td colspan="2">3. Recipient's Catalog No.</td></tr>
<tr><td colspan="3" rowspan="2">4. Title and Subtitle
Heavy Axle Study: Impact of Higher Rail Car Weight Limits on Short Line Railroads, Vol. I: Final Report</td><td colspan="2">5. Report Date
October 29, 2001</td></tr>
<tr><td colspan="2">6. Performing Organization Code</td></tr>
<tr><td colspan="3">7. Author(s)
Jeffrey A. Laman, Charles A. Leighty, Gary L. Gittings, Ece Erdogmus and Thomas E. Boothby</td><td colspan="2">8. Performing Organization Report No.
2001-26-I</td></tr>
<tr><td colspan="3" rowspan="2">9. Performing Organization Name and Address
The Pennsylvania Transportation Institute
Transportation Research Building
The Pennsylvania State University
University Park, PA 16802-4710</td><td colspan="2">10. Work Unit No. (TRAIS)</td></tr>
<tr><td colspan="2">11. Contract or Grant No.
359704 Work Order 27</td></tr>
<tr><td colspan="3" rowspan="2">12. Sponsoring Agency Name and Address
The Pennsylvania Department of Transportation
Bureau of Planning and Research
Commonwealth Keystone Building
400 North Street, 6th Floor
Harrisburg, PA 17120-0064</td><td colspan="2">13. Type of Report and Period Covered
Final Report 10/14/99-5/31/01</td></tr>
<tr><td colspan="2">14. Sponsoring Agency Code</td></tr>
<tr><td colspan="5">15. Supplementary Notes
COTR: Jim Smedley</td></tr>
<tr><td colspan="5">16. Abstract
The current Class I railroad trend is beginning to require that short-line railroads accept heavy-axle cars beyond the previous standard 263,000-lb gross car weight. Given the nature of the short-line railroad infrastructure in Pennsylvania, these heavier cars are expected to be met by under-capacity of both track structures and bridge structures. The economics of short-line railroads are such that it is important that Class I railroads be able to meet this new demand; therefore, the present study has been undertaken to begin the statewide assessment of the short-line railroad ability to support heavier loads.
In the economic interest of the Commonwealth of Pennsylvania, the PENNDOT Bureau of Rail Freight, Ports, and Waterways funded the present study to estimate the cost for a statewide upgrade of the short-line infrastructure to accommodate the 315,000-lb and 286,000-lb gross car weights.
This project investigated the infrastructure of short-line railroads to safely support 315,000-lb and 286,000-lb gross car weights through a bridge statistical sampling scheme and a track survey.

This report is published in two volumes, Volume I: Final Report, and Volume II: Appendices.</td></tr>
<tr><td colspan="3">17. Key Words
Class I railroad, short line railroads, heavy axle, weight limits</td><td colspan="2">18. Distribution Statement
This publication is a confidential document that may not be published, reproduced, released or discussed without the written permission of PENNDOT.</td></tr>
<tr><td>19. Security Classif. (of this report)
Unclassified</td><td colspan="2">20. Security Classif. (of this page)
Unclassified</td><td>21. No. of Pages
49</td><td>22. Price</td></tr>
</table>

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

HEAVY AXLE STUDY: IMPACT OF HIGHER RAIL CAR WEIGHT LIMITS ON SHORT-LINE RAILROADS

University-Based Research, Education and Technology Transfer Program
Agreement No. 359704
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VOL. I: FINAL REPORT

Prepared for

Commonwealth of Pennsylvania
Department of Transportation

By

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October 2001

This work was sponsored by the Pennsylvania Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration. The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of either the Federal Highway Administration, U.S. Department of Transportation, or the Commonwealth of Pennsylvania at the time of publication. This report does not constitute a standard, specification, or regulation.

PTI 2001-26-I

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EXECUTIVE SUMMARY

Motivated by the desire to reduce operating costs and to improve equipment productivity, Class I railroads have increased gross car weights (GCW) from the established standard of 263,000 lb to 286,000 lb. A few are considering even higher GCWs, such as 315,000 lb. Short-line railroads (SLRR) are now under pressure to upgrade their infrastructure to support the new GCWs. Given the nature of the short-line railroad infrastructure in Pennsylvania, these heavier cars are expected to impose higher maintenance costs and possibly require significant upgrading of both track and bridge structure load-carrying capacity.

Many SLRRs, however, operate in an extremely tight profit environment and occasionally require governmental assistance for infrastructure rehabilitation and upgrading. The potential financial impact of higher weight standard rail cars is therefore a concern not only to the SLRRs themselves but also to the federal and state agencies that provide financial assistance. Thus, the fundamental objective of this study, sponsored by the Pennsylvania Department of Transportation's Bureau of Rail Freight, Ports, and Waterways, is to establish and implement a methodology to quantify the financial impact of higher GCWs on the infrastructure load carrying of SLRRs operating within the Commonwealth of Pennsylvania.

METHODOLOGY

A statewide analysis of all SLRR bridges and track structures is not feasible due to the extremely large number of structures (approximately 2,000) and miles of track (approximately 3,000). Given prior research on the impact of higher GCWs on track structure and the constraints on time and monetary resources for this study, the research team developed a stratified random sampling methodology focused on evaluation of bridge structures, with secondary consideration on track impacts. A stratified sampling plan was used to ensure that the sample contained examples of each critical material and bridge type that would significantly affect the financial cost estimate.

Much of the early effort in this study focused on soliciting track and bridge structure data from the SLRRs operating within Pennsylvania. Data provided by 35 SLRRs responsible for 76 branch lines were compiled into bridge and track databases that include adequate detail for 1,200 bridges and 1,600 miles of track structure. The bridge database established the population from which the bridge sample was drawn. Data collected from field work on the bridge sample formed the basis for subsequent detailed structural evaluation and cost analysis. The track database was evaluated in its entirety on the basis of the limited track information that was made available to the research team.

The 25 sampled bridges were evaluated based on field inspections conducted by the research team at each of the 25 bridge sites. The evaluations of the 25-bridge sample were used to establish load-carrying capacity estimates and required strengthening costs for under-capacity structures. These costs were then extrapolated from the sample to the bridge population. Rail costs for the Commonwealth are based on a strengthening and cost evaluation of the entire track structure database compiled from the participating SLRRs and are not extrapolated to a statewide population. Each bridge and mile of track was evaluated for force effects from five different loads: 263,000-lb GCW, 286,000-lb GCW, 315,000-lb GCW, Cooper E-loading, and alternate live loading with final strengthening costs based on the 286,000-lb and 315,000-lb GCW proposals of concern for the present study.

FINDINGS

Twelve of the 25 sample bridges rated at an E80 live load, which is sufficient to support a 315,000-lb GCW. Eight of the 13 remaining sample bridges will safely support each of the three GCW loads, resulting in 5 bridges that will not safely support either the 286,000-lb or the 315,000-lb GCW load. Support of the 286,000-lb and 315,000-lb GCW is the focus of the study and the basis of strengthening schemes.

The 263,000-lb GCW, E-loading, and alternate loading are for reference purposes only.

Based on the bridge structural analysis, five bridges were determined to be under-capacity for both the 286,000-lb and 315,000-lb GCW proposals. Due to this marginal nature of railroad bridge under-capacity, bridge strengthening schemes were developed for each of the five under-capacity bridges. This results in the sample and, therefore, the extrapolated statewide bridge strengthening cost for both 286,000-lb and 315,000-lb GCW to be the same. Cost estimates for each strengthening scheme were obtained from recent similar work conducted in the Commonwealth of Pennsylvania. Upgrade costs for the bridge sample were then extrapolated to the entire population, resulting in an estimated bridge upgrade cost of $8,500,000 for either 286,000-lb or 315,000-lb GCW proposals. The width of the confidence interval is 8% for a confidence of 95%, meaning that the research team is 95% confident that the bridge cost estimate is within the range of $7,820,000 - $9,180,000.

Based on the track structural analyses of the database developed for participating SLRRs, 75 miles of track were determined to be under-capacity for 286,000-lb GCW and 288 miles of track were determined to be under-capacity for 315,000-lb GCW. Strengthening of under-capacity track miles requires rail replacement. The estimated total cost for 75 miles of track replacement (286,000-lb car) is $17,000,000 and for 288 miles (315,000-lb car) is $64,000,000.

1. INTRODUCTION

1.1 BACKGROUND

There are three fundamental paths to improved profitability: cost control, productivity improvement, and revenue growth. The landmark federal economic regulatory reforms of the 1976-1980 period provided the railroad industry with its best opportunity in nearly 90 years to pursue all three profitability paths. Initially, the industry focused its efforts on cost control and productivity improvement. Initiatives included a dramatic increase in Class I (railroads with annual operating revenues exceeding $250 million in 1991 dollars) merger activity, which created opportunities to sell or abandon duplicate rail mainlines and unprofitable low-traffic-density lines, work force reductions, work rule revisions, investments in networking and communications technologies, and investments in rail infrastructure and new equipment. The latter include programs to redesign rail cars for increased capacity and higher utilization.

Until relatively recently, the standard maximum gross weight for a four-axle rail car in the United States was 263,000 lb. However, in the 1980s Class I railroads began pushing the maximum higher to a target of 286,000 lb, or approximately 36 tons per axle. Some carriers have interest in achieving an even higher maximum standard of 315,000 lb, or about 39 tons per axle.

Through internal research and the Heavy Axle Load Research Program operated by the Transportation Technology Center, Inc., a subsidiary of the Association of American Railroads, the Class I rail carriers carefully study the productivity, cost, and overall financial impacts of each proposed change in standard maximum rail car weights. The benefits of higher gross weight cars are reasonable clear. They include equipment capital cost savings from being able to provide the same amount of capacity at a lower cost per ton of car capacity and various operating cost savings, including lower overall car maintenance costs, fewer car miles, fewer crew starts, and less fuel consumption required to move a given volume of traffic[1]. The costs include increased wear on rail, ties, ballast, and subgrade and possible higher capital costs to upgrade track and bridge structures to accommodate the heavier loads.

The magnitude of the latter cost is railroad and route specific. In general, much of the Class I systems were originally built or recently improved with heavy rail and better ties and ballast sufficient to carry heavier loads. These systems are also generally well maintained. In contrast, the systems of short-line and regional railroads consist primarily of former Class I lines originally built to serve relatively small traffic-generating locations. Thus, these lines were generally constructed for lower volumes and lighter-weight traffic. Furthermore, most short-line and regional railroads are very small organizations with relatively modest capitalization. For many, access to outside financial capital is very limited. While most maintain their systems in safe operating condition, they may not have the financial resources to continue maintaining minimum safe standards if the impact of higher-weight rail cars on track and bridge structure substantially raises maintenance costs or requires significant upgrading.

Among those concerned about the impact of heavy-axle cars on short-line and regional railroads are government agencies charged with promoting and facilitating the transport of goods by rail. Recognizing the financial constraints on many short-line railroads, agencies at both the federal and state levels sponsor low-interest loan and grant programs earmarked for short-line rail infrastructure improvement. The Commonwealth of Pennsylvania sponsors one of the largest grant programs to assist its contingent of nearly 70 short-line rail operators. As a state with one of the largest short-line networks in the nation, the Commonwealth recognizes the potentially significant impact heavy-axle rail car operations may have on requests for financial assistance from its short-line railroads. Consequently, in 2000, the Commonwealth sponsored the research effort presented in this report.

1.2 RESEARCH PURPOSE AND OVERVIEW OF RESEARCH METHODOLOGY

The purposes of this study are (1) to determine through structural evaluation whether short-line railroad bridge and track structures within the Commonwealth have the capacity to safely carry higher maximum-gross-weight rail cars, (2) to estimate the magnitude of rehabilitation and/or replacement costs and recommend maintenance schedules for those structures that will not safely accommodate the higher-weight cars, and (3) determine the short-line railroad corridors in Pennsylvania that currently have the most significant heavy-axle rail car flows. While track is considered, the primary focus of the study is on rail bridges, in part because other research on this issue has concentrated on track impacts while no other known research has been conducted on rail bridge impacts.

As the study began, it was generally believed that there were over 2,000 short-line rail bridges in the Commonwealth. Financial and time constraints of the study, however, only permitted structural evaluations for a small sample of 25 bridges. Given the necessity of sampling, it was critical to ensure that the sample was representative of the principal variables that would cause variation in the rehabilitation cost estimate. Aside from the unknown condition of each structure, the research team believed three factors are critical to variation in potential rehabilitation costs: (1) construction material (masonry, stone, concrete, etc.), (2) bridge type (truss, girder, etc.), and (3) length. The research methodology, therefore, provided for a stratified random sampling of the population, with strata defined by construction material and bridge type. The length variable was incorporated into the procedure for estimating the confidence intervals on the cost estimate. Therefore, the methodology consisted of the following steps:

- Develop a definition of a bridge and request rail bridge and track data from the Commonwealth and short-line railroads. Assemble a rail bridge population, restricting the population to short-line rail bridges on rail freight lines within the Commonwealth.

- Construct a representative stratified random sample of the bridge population using construction material and bridge type as strata. Incorporate bridge length in the process for estimating confidence intervals.

- Conduct structural field evaluations of each bridge in the sample. While in the field, observe track and tie conditions on portions of track leading to each structure.

- Analyze the field data and determine bridge and track structures that cannot safely accommodate 286,000-lb and 315,000-lb rail cars. Develop a bridge and track structure condition database.

- Estimate retrofit and upgrade costs for bridges and track structure that cannot safely carry heavier rail cars. These cost estimates are to use unit costs for recent similar construction and strengthening schemes that are typical for the particular structure type.

- Develop recommendations on maintenance schedules.

! Extrapolate the sample cost estimates to the entire short-line bridge and track population to determine an estimate of the total upgrade costs required on Pennsylvania short lines to safely carry heavy-axle rail cars.

1.3 ORGANIZATION OF REPORT

The last portion of this chapter addresses the current heavy rail car flows on short-line corridors in Pennsylvania. Chapter 2 presents the development of the bridge and track population and the content of the resulting databases. The selection of criteria used to stratify the bridge population for sampling is discussed and the distribution of the bridge population by material and bridge type is presented. Chapter 3 covers the rationale for the sampling approach and the sampling procedure itself. Chapter 3 also presents the sample distributions for each stratification and compares each of these to the bridge population distribution. Chapter 4 discusses the field evaluation and analysis procedures and the results for the sampled bridge structures and the track survey. Chapter 5 addresses the development of the cost data and estimates for repair and rehabilitation schemes for structures that were found inadequate for heavy-axle rail cars. Chapter 6 summarizes and concludes the report. Throughout the report, references are made to the 13 appendices that provide additional explanation or data.

1.4 HEAVY-AXLE RAIL CAR FLOWS IN PENNSYLVANIA

One purpose of this research was to determine the current magnitude of heavy-axle rail car flows on short line rail corridors in Pennsylvania. The two principal Class I railroad carriers operating in Pennsylvania, Norfolk Southern Corporation and CSX Transportation, were each requested to provide data on heavy axle rail car interchange volumes with short-line railroads operating in Pennsylvania. Each railroad provided typical monthly car flow data for 286,000-lb and 315,000-lb cars for various periods during 1999 and 2000. Each railroad also provided the names of the short lines with which these cars were interchanged and the junction points of the interchanges.

The heavy axle rail car data are tabulated and presented in Table 1.1. As indicated, the movements are predominantly 286,000-lb cars. To obtain a better geographical picture of these data, the junction points through which these cars are flowing were located on a Pennsylvania railroad map and are presented in Figure 1.1. An abbreviation for the name of each short line railroad involved in heavy-axle car moves is also shown on the map near each respective junction point.

As indicated by the legend, the junction points have been coded on the basis of the relative magnitude of the monthly heavy-axle car flow. The most intense flows are on a northeast-southwest axis that runs through the central part of the state. Secondary intensity is found in the area southeast of Harrisburg. Note that no distinction could be made as to whether the traffic is inbound or outbound to the connecting short line, nor was information obtained on the number of train movements associated with heavy-axle moves at each junction.

1. Zarembski, Allan M. and Jim Blaze, ASmall Railroads and Heavy Loads,@ *Railway Age*, April 2000, pp. 68-69.

Table 1.1 Heavy (286k) Rail Car Flows.

Railroad	Volume (cars/month)	Junctions with Class I Railroads
NSRC: Norfolk Southern Railway Co		
PBNE	180	Bethelhem
EEC	5	Erie
TYBR**	83	Fair
GBRY	5	Gettysburg, Mt Holly Spring
EV	90	Hollidaysburg
HRS	70	Hollidaysburg
RJCP	1320	Keating, Cresson
SBRR	5	Lackawaxen
RBMN*	1850	Lehighton, Mehoopany
JVRR	75	Lewistown
NBER	740	Lock Haven, Tyronne
UCIR	130	Milton
NSHR	100	Northumberland
SVRR	60	Northumberland
SH	750	Steelton, Highspire
TMSS	10	Towanda
TCKR	15	Trafford
YRC	660	York
CSXT: CSX Transportation		
YRC	305	Hannover, Porters
BPRR	1618	New Castle, Erie
WE	1819	Ohiopyle, Brook
KKRR	1	Could not be identified

*** RBMN volumes from 2 sources were (666 vs 1850). Difference could not be reconciled.**
**** TYBR takes the 315k cars.**

Abbreviations

NSRC	Norfolk Southern Railway Co
SVRR	Shamokin Valley RR corp
SBRR	Stourbridge Railroad Co
EEC	East Erie Commercial RR
EV	Everett Railroad Co
GBRY	Gettysburg Railroad Company
HRS	Hollidaysburg and Roaring Spring Railroad Co.
JVRR	Junaita Valley Railroad Co
NBER	Nittanny and Bald Eagle RR Co
NSHR	North Shore Railroad Co
PBNE	Philadelphia Bethelhem and New England RR
RJCP	RJ Corman Railroad Co.
SH	Steelton Highspire RR
TCKR	Turtle Creek Industrial RR
TMSS	Towanda-Monroeton Shippers Lifeline
UCIR	Union County Industrial RR
YRC	York rail company
RBMN	Reading Blue Mt. and Northern Railway
TYBR	Tyburn Railroad Co
CSXT	CSX Transportation
BPRR	Buffallo and Pittsburg Railroad
KKRR	Knox and Kane RR co
WE	Wheeling and Lake Erie Railroad

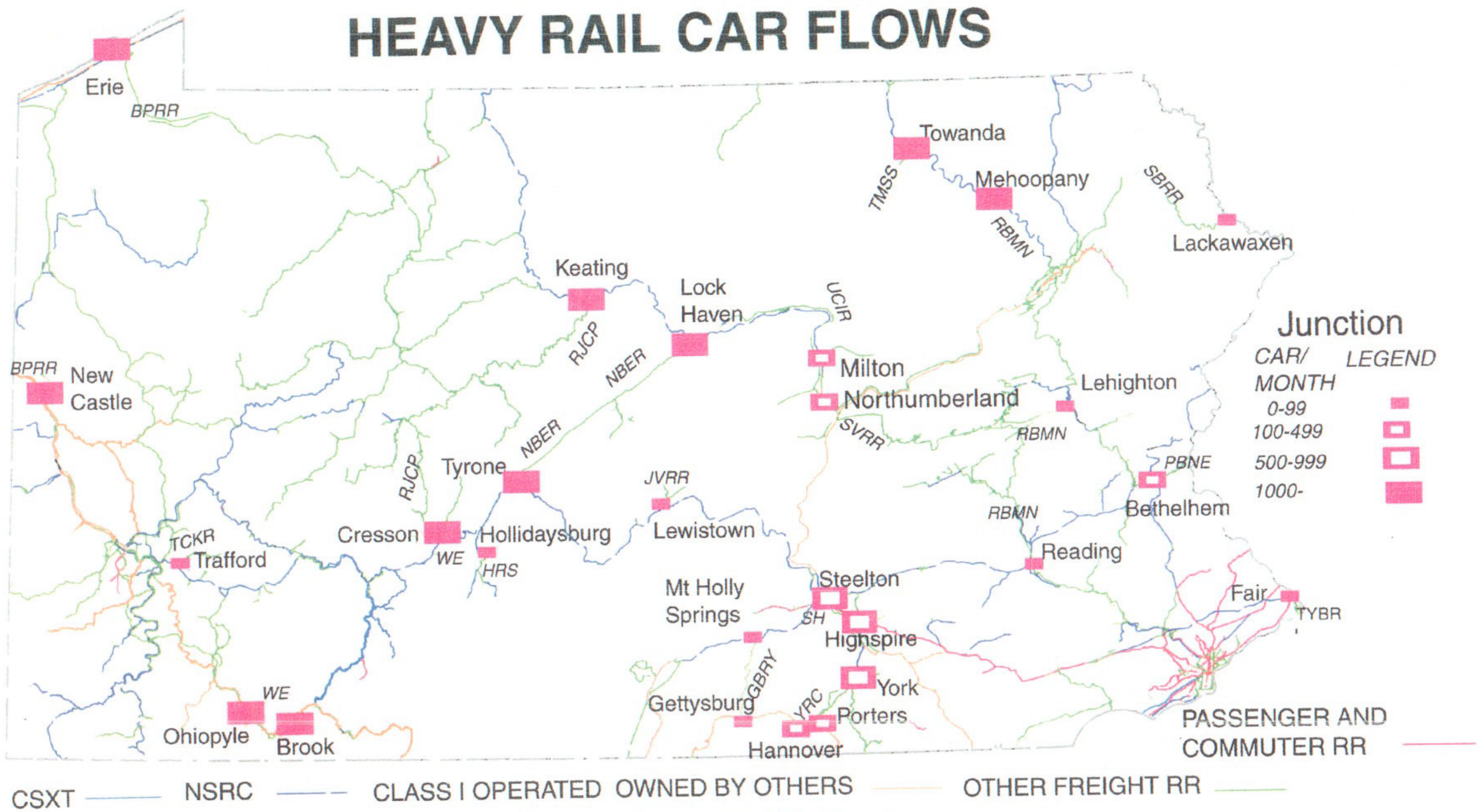

Figure 1.1 - Heavy Axle Rail Car Flow Data

2. BRIDGE AND TRACK DATABASE DESCRIPTION

In order to complete the statistical evaluation of the Pennsylvania short-line bridge population and to evaluate the track structure, two databases were constructed for the present study that consist of information obtained from cooperating Pennsylvania short-line railroads. A bridge structure database that includes all immediately available bridge information from the short-line owners/operators was established for the study. In addition, the short-line owners/operators were surveyed to develop a database of available track structure information. The two databases were developed in Microsoft Access 2000. Following is a description of the data collection procedure and the resulting bridge and track databases.

2.1 BRIDGE DATABASE DESCRIPTION

The bridge database forms the basis for the development of the study population from which the bridge detailed evaluation sample was drawn in order to achieve the objectives of the study. The bridge population was developed with criteria to define bridge structures suitable for the study; therefore, bridges in the database not meeting the criteria were omitted from the population to create the population database used for the sampling procedure.

The database contains all data available from participating short-line owners/operators. Short-line owners/operators typically do not maintain complete infrastructure databases, so the data sets such as milepost (bridge location), bridge condition, rating, type, and others were generally not readily available. In addition, short-line infrastructure information is typically not available from the short-line owners/operators in an electronic format and is incomplete for virtually every bridge.

Initially, bridge structure data were requested from short-line owners/operators via a mailed survey. The survey was initiated to discover the format, completeness, and condition of the information for the bridge and track information maintained by the owners/operators as well as the prevalence of engineering consultant-maintained infrastructure information. It was determined that the mailed survey was not successful due to a 20% return rate. Due to this low survey response, the short-line owners/operators were contacted directly to obtain information. Following an extensive data-collection effort, a final bridge database was established with 1,517 short-line railroad bridges (see appendix A) as a result of an approximately 80% short-line participation rate. The estimated state-wide short-line railroad bridge count is approximately 2,200, resulting in an estimate that approximately 70% of the Commonwealth bridges have been entered into the project database. Of the short-line railroads that provided bridge data, about 15% account for 60% of the bridges in Pennsylvania.

The bridge database was created with 20 data sets, including owner, branch, bridge number, bridge milepost, length, number of spans, deck type, construction material, age, number of tracks, width, what the bridge is over, gross car weight (GCW) (286,000 lb or 315,000 lb), date inspected, condition, replacement cost, history, remarks, and availability of bridge plans. Constructing the database involved a significant data-sorting effort, as 60% of the short-line railroads provided bridge information in non-electronic format that required transfer to the database. Because each short-line owner/operator uses a unique notation for bridge type and condition, a conversion of each system to a master format for the project database was required (see Table 2.1). In addition, several short-line owners/operators defined certain multispan, multi-type bridges as a single bridge type, ignoring the different bridge types and span lengths from abutment to abutment. If a bridge could be classified as more than one type or different span lengths, the bridge was divided into the different types and lengths and then entered into the database separately.

In order to draw the study sample a study population was defined and established. Bridges of the study population had to meet certain specific criteria; bridges not meeting these criteria were excluded from the study population database:

Table 2.1. Bridge Type Codes.

Material	Type	Code
Steel	Deck Plate Girder	DPG
	Through Plate Girder	TPG
	Deck Truss	DTR
	Through Truss	TTR
	Stringer	SST
	Corrugated Metal Pipe	CMP
	Cast Iron Pipe	CIP
	Railtop	TOP
Masonry	Arch	MAR
	Box	MBX
Concrete	Arch	CAR
	Stringer	CST
	Box	CBX
	Slab	CSB
	Pipe	PIP
Timber	Stringer	TST

1. The short-line railroad must haul freight.
2. The short-line bridge must be located in Pennsylvania.
3. All Class 1 railroads are excluded.
4. Short-line railroads previously rated for 315,000-lb GCW are excluded.
5. Culverts and pipes are excluded.

Using these criteria, certain bridges were excluded. One short-line railroad maintains a published clearance for 315,000 lb GCW and was, therefore, excluded from the study population. A second short-line railroad was excluded from the study population because it does not support the hauling of freight. The final legend for the bridge types included in the population is shown in Table 2.2. The established bridge types are also the stratifications for the study's random stratified sampling. The final short-line railroad bridge population contains 1,174 bridges and is provided in appendix B.

Table 2.2. Bridge Stratifications and Legend.

Material	Type	Legend	Quantity
Steel	Deck Plate Girder	DPG	467
	Through Plate Girder	TPG	140
	Deck Truss	DTR	34
	Through Truss	TTR	36
	Stringer	SST	126
	Railtop	TOP	10
Masonry	Arch	MAR	91
Concrete	Arch	CAR	98
	Stringer	CST	7
	Slab	CSB	129
Timber	Stringer	TST	36
Population Total			1,174

2.2 TRACK DATABASE DESCRIPTION

Short-line track structure data were collected in conjunction with the bridge data collection effort from the short-line owners/operators. Due to study constraints, inspection of the track structure was limited to the sections of track directly adjacent to the study sample bridges. Track structure data sets included in the short-line track database include: operational track length, rail size and condition, Federal Railroad Administration (FRA) or operation speed class, condition and spacing of ties, and depth of ballast. These data sets are very incomplete due to the lack of information available from the short-line owner/operators. Practically no information is available regarding the existing ballast, which is needed for a complete analysis of the track structure by the procedures discussed in appendix M. The length of a given rail size and operation speed information were obtained for the entire database. In addition, rail condition was also available, which did allow rail strength evaluation. A total of 1,505 miles of track are contained in the project database. The track structure inspection notes are summarized in appendix H.

3. DEVELOPMENT OF THE BRIDGE SAMPLING PROCEDURE

3.1 SAMPLING PHILOSOPHIES AND APPROACH

There are a number of possible sampling approaches that may be used to accommodate the difficulty of evaluating an extremely large population; however, the general statistical philosophy behind the various sampling approaches is based on the same principles. The philosophy of random sample evaluation assumes that through the investigation of a statistically significant and randomly selected, or unbiased, subset of the population, the entire population can be represented. Because the economic advantages of investigating a population subset of the entire population are so great, this philosophy is used by many disciplines. Sampling is required for the present study due to resource and time constraints. Careful evaluation of the statewide short-line railroad bridge population would require that approximately 2,000 structures be individually evaluated, which clearly is not feasible; thus the design of a sampling methodology has been undertaken.

It is desirable to establish a bridge sample that is as representative a cross section of the total population as possible and yet eliminate any bias from the sampling methodology. Several methods exist to develop a randomly selected, unbiased, statistically significant sample of a population. The simplest method randomly selects a subpopulation from the population to be investigated. This method may fall short of the needs of the present study, as it may not establish a sample that is a representative cross section of the population. A more powerful sampling method is known as stratified random sampling, which requires a data set from which the population can be divided into groups, or strata. This method causes the sample to better represent the population if the stratifying data set directly affects the evaluation of interest. This method seeks a matching of the proportion of each stratum in the population to the proportion of that stratum in the sample. Equation 3.1 determines the number of data points that must be taken from each stratum, n_h:

$$n_h = \frac{n * N_h}{\sum_{h=1}^{L} N_h} \tag{3.1}$$

where n is the sample size, N_h is the number of bridges in the h stratum and L is the number of strata. A variation of this method employs a third data set that is related to information of interest. This approach can better represent the population by pulling a sample that is proportional to the population as a result of selecting the stratum, which is then weighted by the third data set. Equation 3.2 illustrates the use of a third data set to select the number of data points from each stratum, n_h:

$$n_h = \frac{n * N_h * \sigma_h}{\sum_{h=1}^{L} N_h \sigma_h} \tag{3.2}$$

where n is the sample size, N_h is the number of bridges in the h stratum, σ_h is the standard deviation of the third data set within the h stratum, and L is the number of strata.

3.2 CONFIDENCE INTERVAL

It has been assumed that all variables follow a standard normal distribution. Because the population of bridges is very large, confirmation of the actual distribution would require an extensive effort; however, a standard normal distribution is reasonable due to the theorem of central tendency. While it is necessary to establish a sample of the bridge population, the accuracy of the information decreases as the size of the sample decreases. The accuracy of the sample in predicting a particular outcome for the population is described by the width of the confidence interval. The confidence interval relates to the probability that the true result for the population is within the confidence width. A wider confidence interval in the present study can be interpreted to correspond to a wider margin of error in the final estimate of bridge upgrade cost as a result of increased rail car weights. The equation for the width of the confidence interval is:

$$\hat{t} \pm t\sqrt{est.\mathrm{var}(\hat{t})} \tag{3.3}$$

where t is the upper $\alpha/2$ of the normal distribution, $est.\mathrm{var}(\hat{t})$ is the variance of the cost of the total population, and α is percent remaining from the confidence interval. The basic equation for the variance of the cost in the total population is:

$$\mathrm{var}(\hat{t}) = \sum_{i-1}^{L} N_h (N_h - n_h) \frac{\sigma_h^{\ 2}}{n_h} \tag{3.4}$$

where σ_h^2 is the variance of the cost for each stratum.

If the strata sizes are greater than 30, a standard normal approximation can be used. If the sample size is less than 30, a t-distribution must be used. Unlike the standard normal distribution approximation, the t-distribution is dependent on a degree of freedom to determine the end value. The equation for the degree of freedom is:

$$d = \left(\sum_{h=1}^{L} a_h \sigma_h^2 \right)^2 \Bigg/ \left[\sum_{h=1}^{L} \left(a_h \sigma_h^2 \right)^2 \Big/ \left(n_h - 1 \right) \right] \tag{3.5}$$

where $a_h = N_h (N_h - n_h) / n_h$.

Equations 3.3, 3.4, and 3.5 indicate that there is no direct relationship between the confidence interval and the size of the sample. The two attributes along with the standard deviation of the cost for the sample are combined to create the width of the confidence interval (CI). The width of the CI for the present study is reported in terms of a cost. This information shows that although there is no direct relationship between confidence interval and sample size, the equations combined show that the larger the sample size the better representation of the overall population. This sample size will need to be as large as possible within the realm of the work in the project.

3.3 PROJECT SAMPLING PROCEDURE

The project sampling procedure consists of establishing strata, establishing a weighting variable, and then pulling a statistically significant, random, unbiased bridge sample from the short-line bridge population in order to evaluate each sample bridge for capacity. The present project employed the method described above, utilizing equation 3.2 in order to establish the most representative sample. The stratifications, which divide the population into 11 strata, are the bridge types listed in Table 2.2. The third data set, or weighting data set, is the bridge length. The bridge length data set was chosen because the length strongly affects the cost of upgrade, as shorter spans may require less extensive reinforcement than longer spans. Both data sets (bridge type and bridge length) are strongly and positively correlated with the variable of interest—that is, bridge strengthening cost, which is the major selection criterion. Many factors influence strengthening cost and are correlated to the variable of interest, including type, material, age, deck width, number of tracks, and deck type. However, complete data sets are not available for any of these considered factors and therefore could not be used in the present study.

Two levels of stratification are identified for the present study. The first level of stratification category is the bridge material, consisting of steel, timber, concrete, and masonry. The second stratification is the bridge type, which consists of arch, girder, truss, and stringer. The third data set, as a weighting factor, is bridge length.

These data sets are defining divisions that directly affect bridge strengthening cost. The distribution of the strata in the population is shown in Table 3.1. Column 6 is the standard deviation of the length for each stratum. This information was required to determine the number of bridges to be sampled from each stratum using equation 3.2.

Table 3.1 Distribution of Strata in Bridge Population.

Material (1)	**Type** (2)	**Legend** (3)	**Quantity** (4)	**Percent** (5)	**σ_h (ft)** (6)
Steel	Deck Plate Girder	DPG	467	40%	163.27
	Through Plate Girder	TPG	140	12%	126.4
	Deck Truss	DTR	34	3%	227.1
	Through Truss	TTR	36	3%	242.1
	Stringer	SST	126	11%	51.7
	Railtop	TOP	10	1%	23.01
Masonry	Arch	MAR	91	8%	93.51
Concrete	Arch	CAR	98	8%	21.79
	Stringer	CST	7	1%	25.9
	Slab	CSB	129	11%	16.86
Timber	Stringer	TST	36	3%	180.1
Population Total			1,174	100%	145.88

The confidence interval procedure indicates that the width of the confidence interval is directly related to sample size. After the sample size was chosen, the stratum sample size was determined using equation 3.2. Results were rounded to integer bridge quantities to randomly sample from each stratum.

Table 3.2 Sample Size and Stratum Distribution.

Stratum (1)	Results (2)	Quantity (3)	Percent (4)
Deck Plate Girder	13.953	14	56%
Through Plate Girder	3.238	3	12%
Deck Truss	1.413	1	4%
Through Truss	1.595	2	8%
Stringer	1.192	1	4%
Railtop	0.042	0	0%
Masonry Arch	1.557	2	8%
Concrete Arch	0.390	0	0%
Concrete Stringer	0.033	0	0%
Concrete Slab	0.398	1	4%
Timber Stringer	1.186	1	4%

Using Minitab Release 13.1 Software Package, the actual 25-bridge sample was pulled from the bridge population (described in Table 3.1) based on the quantities presented in Table 3.2, column 3. The SLRR bridge sample used for detailed evaluation is presented in appendix C.

4. DETAILED EVALUATION OF THE BRIDGE SAMPLE AND TRACK SURVEY

4.1 BRIDGE EVALUATION RESULTS

Each of the 25 sample bridges was inspected during the months of August and September 2000. Inspections focused on bridge structural information necessary to evaluate the member and bridge strength. The 25 bridges were evaluated for degree of corrosion of the primary structural members. Generally, end spans were inspected because the environment and length are approximately the same for end spans as interior spans, but allowed easier access. The 25 bridges were evaluated for five different live loadings, as detailed in appendix K:

1. Cooper loading
2. Alternate live load
3. 263,000-lb GCW
4. 286,000-lb GCW
5. 315,000-lb GCW

Eight bridge types and four different construction materials are represented in the sample. Each bridge type must satisfy respective evaluation criteria for the various members in the structure and is rated according to American Railway Engineering Association (AREA) 1996 specifications. Standard analysis techniques were employed wherever possible. A commercially available computer analysis program was used to model the behavior of a number of the more complex, statically indeterminate structures. The finite element software package ANSYS (version 5.6) was used for modeling and analyzing the masonry arch bridges.

Individual bridge evaluation results are presented in Table 4.1. A highlighted row indicates that a bridge is not adequate to support a minimum of 315,000-lb GCW. Five bridges do not have sufficient capacity to support either the 286,000-lb GCW or the 315,000-lb GCW, and four bridges will not support the 263,000-lb GCW (see Table 4.1, columns 5, 6, and 7). The five bridges included three different material types: masonry, timber, and steel, and four different bridge types: through-plate girder, through-truss, timber stringer, and masonry arch. Highlighted bridges in column 8 of Table 4.1 indicate rating values below the AREA E80 loading. Of the 25 bridges in the sample, 12 meet the AREA E80 loading criterion. Figure 4.1 presents a histogram of the number of bridges in each E rating increment of 5 kips. Of the 13 bridges that do not meet the AREA E80 loading criterion, five are unable to support the 286,000-lb and 315,000-lb GCW loading.

Table 4.1 Evaluation Results.

BR # (1)	Spans (2)	Length (ft) (3)	Type (4)	263 (5)	286 (6)	315 (7)	Cooper E (8)	Alt E (9)
1	4	280.0	DPG	135%	126%	123%	69.9	76.7
2	6	298.5	DPG	155%	144%	138%	80.0	87.5
3	1	91.5	DPG	135%	126%	141%	73.8	101.3
4	1	48.0	TPG	99%	92%	87%	56.1	48.1
5	1	40.0	DPG	205%	189%	173%	119.1	106.6
6	1	55.0	DPG	171%	159%	149%	91.5	95.9
7	1	80.0	DPG	164%	153%	157%	91.0	118.1
8	1	24.0	TPG	89%	83%	77%	49.1	42.3
9	1	33.5	SST	247%	240%	233%	326.6	286.7
10	1	45.0	TPG	137%	130%	122%	85.5	81.1
11	1	31.0	DPG	116%	107%	102%	67.2	58.3
12	1	72.0	DPG	157%	146v	157%	93.2	111.1
13	1	95.0	DPG	158%	148%	162%	92.5	134.7
14	2	151.0	DPG	213%	199%	215%	119.5	150.7
15	1	100.0	DTR	114%	107%	121%	66.0	95.0
16	1	152.0	TTR	110%	103%	105%	62.7	89.0
17	1	10.0	MAR	133%	132%	132%	363.5	2253.8
18	1	16.0	MAR	40%	35%	35%	20.0	18.0
19	1	9.0	TST	64%	59%	55%	39.5	31.5
20	1	7.0	CSB	122%	113%	108%	73.9	59.1
21	1	45.0	DPG	153%	144%	138%	92.4	93.1
22	18	1290.0	DPG	131%	126%	129%	91.8	126.1
23	4	181.0	DPG	146%	136%	136%	79.4	86.7
24	1	150.5	TTR	101%	93%	84%	64.2	66.5
25	2	120.0	DPG	109%	102%	113%	56.3	73.6

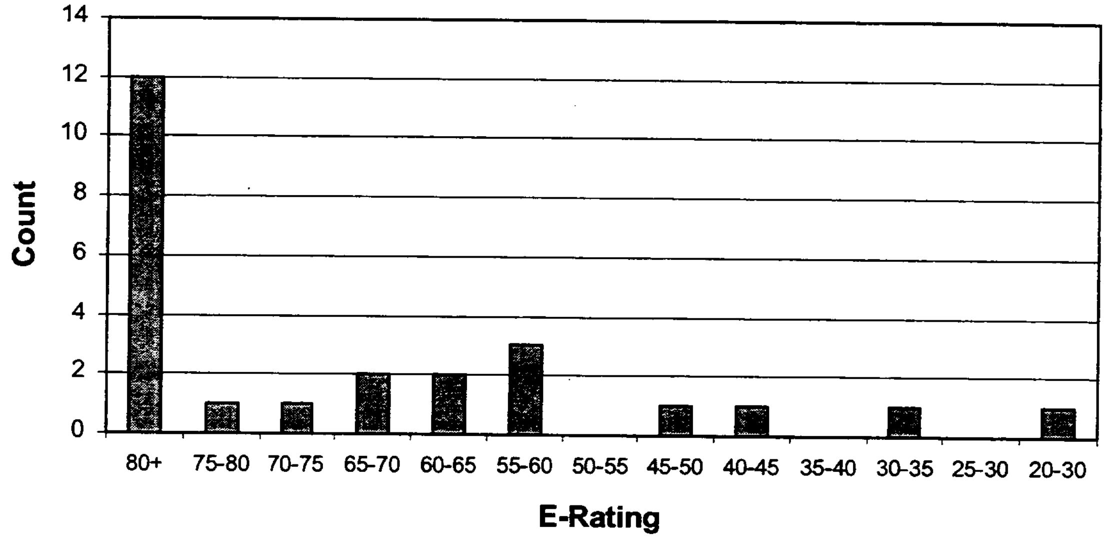

Figure 4.1 Number of Bridges in AREA E-Rating Group.

A number of bridges were determined to possess load capacity significantly in excess of that required to support the 315,000-lb GCW due to past repairs, strengthening, or other circumstances. As an example of other circumstances, Bridge #14 has four girders supporting a single track; however, the structure previously supported two tracks. When one track was removed, all girders were moved to support the remaining single track. Another example is a masonry arch structure, Bridge #17, that exceeds the needed capacity due to the high dead-to-live-load ratio of the structure.

4.1.1 Review of Under-Capacity Bridges

Bridge #4 was designed to support two tracks. However, one track was removed at some unknown date in the past. The structural system consists of three main girders that span 45 ft-9 in and simply supported stringers at 5 ft-0 in, which is approximately the distance between the centerlines of the two rails. The track has shifted 8-in off the center line of the stringers, which changes the live-load proportion for the stringer to 62.1% from the expected 50% in the original design. This shifting of the track causes the stringer to rate at only 87% of the required AREA moment capacity; therefore, the bridge was rated at E48.

Bridge #8 was designed to support two tracks, but one track has been removed. The bridge is a through-plate girder with three main girders, with the tracks located between girders. Floor-beams and stringers support the ballasted tie deck. The rating is governed by the floor-beams, which have 76% of the shear capacity and 93% of the moment capacity required to support the 315,000-lb GCW.

Bridge #18 is a masonry arch bridge with a span of 16 feet. A finite element analysis using the ANSYS software package was performed to determine the applied stresses. One spandrel wall has collapsed and the spandrel wall analysis indicates serious potential for cracking due to high tensile stresses created by transverse soil pressure.

Bridge #19 is a timber bridge that supports each rail with three 8-in-by-16-in timber stringers. Timber stringers were assumed to be red oak in order to determine allowable stresses. The stringers have 55% of the required AREA shear capacity and are also inadequate in bending, resulting in a rating of E31.5. Additionally, Bridge #19 has a severe tie-spacing problem due to the lack of tie spacers, resulting in the ties wandering and opening a clear distance between ties of 26-in.

Bridge #24 is a steel through-truss structure. All members rated sufficient for the 315,000-lb GCW; however, the ties do not meet the AREA strength requirements. Three adjacent stringers spaced at 42-in center-to-center support the ties. The applied stresses were evaluated using SAP2000 to model the bending of the rail, ties, and stringers simultaneously.

4.1.2 Under Capacity Bridge Strengthening

The goal of establishing strengthening schemes and corresponding costs is to evaluate the impact of heavier GCW (286,000-lb and 315,000-lb) rather than to evaluate the impact of other loads such as E80 or the alternate loading. E80 and alternate loading results are included herein for the purpose of reference only and are not the focus of the present study. Each of the five under-capacity bridges was evaluated for strengthening adequate to support the 286,000-lb and 315,000-lb GCW loading, and several schemes were developed for each and investigated to determine the most cost-effective. The strengthening costs were then determined by four separate contractors that assembled estimates for each of the five final strengthening schemes.

Bridge #4: The rating controlling stringer consists of a built-up section made of a web plate and riveted angles as flanges. An ASTM A36 steel plate, ½-in-by-13-in-by7-ft-0-in added to the flange adequately strengthens the stringer. Realignment of the track to its original position was considered as a strengthening scheme; however, the track needs to be moved 8-in to alleviate the overstress in the stringers over an extensive distance. This scheme was judged to be cost ineffective.

Bridge #8: The rating controlling floor-beam requires additional shear and bending capacity. A strengthening scheme was investigated consisting of bolting plates to the web plate of the floor beam. This requires that bolts be placed on a 3-in-by-3-in grid for proper load transfer between plates. A ¼-in-by-13in-by-48-in steel plate installed between the stringer and main girder on one side of the floor beam adequately strengthens the member. To decrease the applied stresses in the member top flange, a post-tensioned rod attached to the bottom of the floor beam has been designed. A 1-in Gr. 270 circular bar placed below the bottom flange and a jacking force of 108 kips relieves the bending stresses. Other schemes were investigated; however, none approached the cost effectiveness of the above-described scheme.

Bridge #18: This bridge is a masonry arch with an overstressed spandrel wall. To alleviate the pressure on the wall, the fill behind the wall must be removed at an angle so the slope will remain stable. In addition, gravel placement behind the spandrel wall was designed to relieve the water pressure and cohesive soil forces on the wall.

Bridge #19: Bridge #19 is significantly under capacity. This timber bridge requires a doubling of the number of stringers according to allowable AREA stresses. Two strengthening schemes were investigated for this bridge. The first scheme attaches four C15x33.9 steel channels to the existing timber stringers. The channels must be bolted at 12-in spacing center-to-center to connect the channels to the wood beams. The second strengthening scheme is to replace the timber stringers with glued-laminated timber members. The design calls for each existing timber stringer to be replaced by a 9¼-in-by-15¾-in glued-laminated timber member.

Bridge #24: This bridge had an insufficient number of ties. The existing ties did not have the area required to alleviate shear forces. The ties are spaced at 15-in center-to-center. The ties had to be spaced on 12-in centers. The strengthening, therefore, is to add one tie with the same cross-section every four and move the other four appropriately to maintain the spacing.

The five bridges described above will not currently support either the 286,000-lb or the 315,000-lb GCW loading; therefore, the strengthening schemes were developed. The above-described strengthening schemes were used to estimate strengthening costs in chapter 5.

4.2 DETAILED EVALUATION OF THE TRACK SURVEY

The SLRR track structure on and adjacent to the 25 inspected bridges (discussed above) was also inspected and evaluated according to the FRA tie-spacing limits and tie conditions (appendix H). In this section, the evaluation of the general track data from appendix G is summarized. Where adequate engineering information was obtained for SLRR track structures, the track structure was evaluated based on AREA specifications. Limited sections of track were found to be insufficient to carry either the current or the proposed car weight limits due to undersized rail. Other sections were determined to be inadequate to support a 286,000-lb GCW and a significant number of track miles were determined to be inadequate to support a 315,000-lb GCW. A track structure evaluation summary is presented in Tables 4.2 through 4.6 with additional detailed tables presented in appendix I.

Track structure capacity is primarily a function of rail strength or size. However, the operational speed limits (FRA classes) and the specific car loading may also change the capacity of the rail section. Tables 4.2, 4.3 and 4.4 present the capacity of common rail sizes according to FRA speed classes 1, 2, and 3 for GCWs of 263,000, 286,000, and 315,000 lbs, respectively. The highlighted cells indicate under-capacity rail sections for the corresponding operation speed (class) and GCW. Summarizing the results of Appendix G, of the 1,505 track miles contained in the database compiled from participating SLRRs, 75 miles of track are inadequate to support a 286,000-lb GCW and 288 miles of track are inadequate to support a 315,000-lb GCW. An under-capacity track summary as a function of rail size is presented in Tables 4.5 and 4.6 for 286,000-lb and 315,000-lb GCWs, respectively. The full analysis detail is provided in Appendix I.

For the purposes of the present study cost estimates, the rail-strengthening scheme is to replace the under-capacity rail. It is assumed that the operational class of the line will remain as it currently is. Because a field inspection of all SLRR track miles was beyond the scope of the present project, the tie condition is not well known. An average tie condition is assumed and no tie or subgrade replacement is included in the cost analysis procedure. Cost estimation for the replacement of the under-capacity rail is discussed in section 5.2. On the basis of the stated assumptions, all 85-lb to 115-lb rail must be replaced. In addition, particular sections of higher-weight rail must be replaced depending on existing SLRR class and GCW under consideration. For details, refer to Tables 4.3, 4.4, and 4.5. Welded 136-lb rail is the normal SLRR rail replacement of choice based on availability, cost, and future capacity and was therefore used in all cases where rail was to be replaced for strength considerations, regardless of theoretical strength requirements.

Table 4.2 Track Evaluation Results for 263,000-lb car (k=2500, all.stress =25,000 psi).

Ref.	Rail Type	Class 1 (10 mph)				Class 2 (25 mph)				Class 3 (40 mph)			
		Rail Capacity	Tie Plates	Tie Bearing Pressure	Required Ballast Depth	Rail Capacity	Tie Plates	Tie Bearing Pressure	Required Ballast Depth	Rail Capacity	Tie Plates	Tie Bearing Pressure	Required Ballast Depth
AREA 1996[1]	115RE	146.37%	155.36%	181.77%	13	130.00%	137.98%	161.44%	14	116.92%	124.10%	145.19%	15
	119RE	148.99%	158.50%	185.45%	13	132.32%	140.77%	164.70%	14	119.01%	126.61%	148.13%	15
	132RE	170.32%	166.96%	195.34%	12	151.26%	148.28%	173.49%	13	136.05%	133.36%	156.04%	14
	133RE	167.84%	166.15%	194.39%	12	149.06%	147.56%	172.65%	13	134.07%	132.71%	155.28%	14
	136RE	171.87%	169.87%	198.75%	12	152.65%	150.87%	176.52%	13	137.29%	135.69%	158.76%	14
	140RE	173.16%	170.63%	199.64%	12	153.79%	151.55%	177.31%	13	138.31%	136.30%	159.47%	14
AREA 1929[2]	90RA-A	115.52%	136.00%	159.12%	14	102.60%	120.79%	141.32%	16	92.28%	108.63%	127.10%	17
	100RE	127.86%	144.27%	168.79%	14	113.56%	128.13%	149.91%	15	102.13%	115.24%	134.83%	16
	110RE	139.17%	149.82%	175.29%	13	123.60%	133.06%	155.68%	14	111.16%	119.68%	140.02%	16
	120RE	153.28%	156.35%	182.93%	13	136.14%	138.86%	162.47%	14	122.44%	124.89%	146.12%	15
	130RE	163.50%	161.73%	189.23%	12	145.21%	143.64%	168.06%	14	130.60%	129.19%	151.15%	15
	140RE	175.49%	167.57%	196.06%	12	155.86%	148.83%	174.13%	13	140.18%	133.85%	156.61%	14
	150RE	200.89%	180.88%	211.63%	11	178.42%	160.65%	187.96%	12	160.47%	144.49%	169.05%	13
ASCE[3]	80ASCE	98.14%	125.88%	147.27%	15	87.16%	111.79%	130.80%	17	78.39%	100.55%	117.64%	18
	85ASCE	98.77%	127.72%	149.43%	15	87.72%	113.43%	132.71%	16	78.90%	102.02%	119.36%	18
Carnegie III[4]	105	111.90%	132.05%	154.50%	14	99.38%	117.28%	137.22%	16	89.38%	105.48%	123.41%	17

[1] AREA Manual, 1996

[2] AREA Manual, 1929

[3] American Institute of Steel Construction

[4] American Institute of Steel Construction

Table 4.3 Track Evaluation Results for 286,000-lb car (k=2500, all.stress =25,000 psi).

Ref.	Rail Type	Class 1 (10 mph)				Class 2 (25 mph)				Class 3 (40 mph)			
		Rail Capacity	Tie Plates	Tie Bearing Pressure	Required Ballast Depth	Rail Capacity	Tie Plates	Tie Bearing Pressure	Required Ballast Depth	Rail Capacity	Tie Plates	Tie Bearing Pressure	Required Ballast Depth
AREA 1996[1]	115RE	134.60%	142.86%	167.15%	14	119.54%	126.88%	148.45%	15	107.52%	114.12%	133.52%	16
	119RE	137.00%	145.76%	170.53%	13	121.68%	129.45%	151.46%	15	109.44%	116.43%	136.22%	16
	132RE	156.62%	153.53%	179.63%	13	139.10%	136.36%	159.54%	14	125.10%	122.64%	143.49%	15
	133RE	154.34%	152.78%	178.76%	13	137.08%	135.69%	158.76%	14	123.29%	122.04%	142.79%	15
	136RE	158.05%	156.21%	182.77%	13	140.37%	138.74%	162.32%	14	126.25%	124.78%	145.99%	15
	140RE	159.23%	156.91%	183.59%	13	141.42%	139.36%	163.05%	14	127.19%	125.34%	146.65%	15
AREA 1929[2]	90RA-A	106.23%	125.06%	146.32%	15	94.35%	111.07%	129.96%	17	84.85%	99.90%	116.88%	18
	100RE	117.58%	132.66%	155.22%	14	104.42%	117.82%	137.85%	16	93.92%	105.97%	123.98%	17
	110RE	127.97%	137.78%	161.20%	14	113.66%	122.36%	143.16%	15	102.22%	110.05%	128.76%	17
	120RE	140.96%	143.78%	168.22%	14	125.19%	127.69%	149.40%	15	112.59%	114.85%	134.37%	16
	130RE	150.35%	148.73%	174.01%	13	133.54%	132.09%	154.54%	14	120.10%	118.80%	139.00%	16
	140RE	161.38%	154.10%	180.29%	13	143.33%	136.86%	160.12%	14	128.91%	123.09%	144.01%	15
	150RE	184.73%	166.34%	194.61%	12	164.07%	147.73%	172.84%	13	147.56%	132.87%	155.45%	14
ASCE[3]	80ASCE	90.24%	115.75%	135.43%	16	80.15%	102.80%	120.28%	18	72.08%	92.46%	108.18%	19
	85ASCE	90.83%	117.45%	137.41%	16	80.67%	104.31%	122.04%	17	72.55%	93.81%	109.76%	19
Carnegie III[4]	105	102.90%	121.43%	142.08%	15	91.39%	107.85%	126.18%	17	82.19%	97.00%	113.49%	19

[1] AREA Manual, 1996

[2] AREA Manual, 1929

[3] American Institute of Steel Construction

[4] American Institute of Steel Construction

Table 4.4 Track Evaluation Results for 315,000-lb car (k=2500, all.stress =25,000 psi, tie spacing=21 in).

Ref.	Rail Type	Class 1 (10 mph)				Class 2 (25 mph)				Class 3 (40 mph)			
		Rail capacity	Tie Plates	Tie Bearing Pressure	Required Ballast Depth	Rail Capacity	Tie Plates	Tie Bearing Pressure	Required Ballast Depth	Rail Capacity	Tie Plates	Tie Bearing Pressure	Required Ballast Depth
AREA 1996[1]	115RE	122.21%	129.71%	151.76%	15	108.54%	115.20%	134.79%	16	97.62%	103.61%	121.23%	18
	119RE	124.39%	132.34%	154.83%	14	110.48%	117.53%	137.51%	16	99.36%	105.71%	123.68%	17
	132RE	142.20%	139.40%	163.10%	14	126.29%	123.80%	144.85%	15	113.59%	111.35%	130.28%	17
	133RE	140.13%	138.72%	162.30%	14	124.46%	123.20%	144.14%	15	111.94%	110.81%	129.64%	17
	136RE	143.50%	141.83%	165.94%	14	127.45%	125.96%	147.38%	15	114.62%	113.29%	132.55%	16
	140RE	144.57%	142.47%	166.69%	14	128.40%	126.53%	148.04%	15	115.48%	113.80%	133.15%	16
AREA 1929[2]	90RA-A	96.45%	113.55%	132.85%	16	85.66%	100.85%	117.99%	18	77.04%	90.70%	106.12%	20
	100RE	106.75%	120.45%	140.93%	16	94.81%	106.98%	125.16%	17	85.27%	96.21%	112.57%	19
	110RE	116.19%	125.09%	146.36%	15	103.20%	111.10%	129.98%	17	92.81%	99.92%	116.91%	18
	120RE	127.98%	130.54%	152.73%	15	113.66%	115.94%	135.65%	16	102.23%	104.27%	122.00%	18
	130RE	136.51%	135.03%	157.99%	14	121.24%	119.93%	140.32%	16	109.04%	107.86%	126.20%	17
	140RE	146.52%	139.91%	163.69%	14	130.13%	124.26%	145.38%	15	117.04%	111.76%	130.76%	17
	150RE	167.73%	151.02%	176.70%	13	148.96%	134.13%	156.93%	14	133.98%	120.63%	141.14%	16
ASCE[3]	80ASCE	81.94%	105.10%	122.96%	17	72.77%	93.34%	109.21%	19	65.45%	83.95%	98.22%	21
	85ASCE	82.47%	106.63%	124.76%	17	73.24%	94.71%	110.81%	19	65.87%	85.18%	99.66%	21
Carnegie III[4]	105	93.42%	110.25%	129.00%	17	82.97%	97.92%	114.57%	18	74.63%	88.07%	103.04%	20

[1] AREA Manual, 1996

[2] AREA Manual, 1929

[3] American Institute of Steel Construction

[4] American Institute of Steel Construction

Table 4.5 Track Evaluation Results for 286,000-lb Car.

Rail Type	80	85	90	100	101	105	110	112	115	119	125	127	130
Miles/Rail Type	6	54	43	169.15	27	36	25.3	119.9	191.5	0	0	91	305.4
Percent of Total	0.40%	3.59%	2.86%	11.24%	1.79%	2.39%	1.68%	7.96%	12.72%	0.00%	0.00%	6.05%	20.29%
Miles of Under Capacity Rail	6	54	15	0	0	0	0	0	0	0	0	0	0

Rail Type	131	132	133	135	136	140	152	155	157	Total (miles)	1505.0
Miles/Rail Type	231.2	100	20.3	1	4.2	22.1	2	44	12		
Percent of Total	15.36%	6.64%	1.35%	0.07%	0.28%	1.47%	0.13%	2.92%	0.80%	Under-capacity (miles)	75.00
Miles of Under Capacity Rail	0	0	0	0	0	0	0	0	0	Percent	4.98%

Table 4.6 Track Evaluation Results for 315,000-lb Car.

Rail Type	80	85	90	100	101	105	110	112	115	119	125	127	130
Miles/ Rail Type	6	54	43	169.15	27	36	25.3	119.9	191.5	0	0	91	305.4
Percent of Total	0.40%	3.59%	2.86%	11.24%	1.79%	2.39%	1.68%	7.96%	12.72%	0.00%	0.00%	6.05%	20.29%
Miles of Under Capacity Rail	6	54	43	29.05	27	36	0	47	46	0	0	0	0

Rail Type	131	132	133	135	136	140	152	155	157	Total (miles)	1505.0
Miles/ Rail Type	231.2	100	20.3	1	4.2	22.1	2	44	12		
Percent Of Total	15.36%	6.64%	1.35%	0.07%	0.28%	1.47%	0.13%	2.92%	0.80%	Under-capacity (miles)	288.05
Miles of Under Capacity Rail	0	0	0	0	0	0	0	0	0	Percent	19.14%

5. COST ESTIMATE FOR BRIDGE SAMPLE AND TRACK SURVEY

The determination of statewide reinforcing costs for short-line railroad bridges and estimates of track structure upgrade involves two separate procedures as a result of the discrete nature of bridges and the continuous nature of track structure. Because the bridge evaluation is based on a sample of 25 bridges, the bridge population cost estimate must be extrapolated from this sample. The track structure, however, is based on a survey of the short-line track infrastructure; therefore, the track is analyzed directly without extrapolation and is based solely on the participating SLRRs. The following sections detail the cost analysis for both bridge reinforcing and track upgrades.

5.1 BRIDGE POPULATION COST ESTIMATE

A statewide upgrade of deficient short-line bridges to support heavier axles would entail a multistage engineering and construction process. These stages are defined herein as distinct engineering functions, including construction. Within each stage, specific tasks associated with either engineering costs or construction costs are identified. The stage task costs are then determined and detailed for each stage, followed by extrapolation of the estimated sample costs to the population. The stage costs extrapolated for the population are then summed to determine the total statewide upgrade cost estimate for short-line bridges. Finally, the confidence width for the statewide bridge upgrade cost estimate is evaluated.

5.1.1 Bridge Upgrade Stages

The engineering stages defined for a statewide upgrade are carefully defined to allow for the determination of an accurate population cost estimate. A statewide upgrade of the population of bridges would contain the following five stages:

1. <u>Site Visits and Bridge Inspections.</u> Because short-line railroads do not maintain up-to-date engineering information for bridges, each structure requires a minimum level

of inspection to ensure an accurate basis for evaluation. Certain existing site conditions may directly affect the load-carrying capacity of a bridge, such as damage due to a vessel collision, erosion due to flooding, corrosion, scour at foundations, or other structural deterioration. This inspection stage, therefore, is required to gather all of the appropriate engineering information to evaluate bridge capacity. It is anticipated that short-line owners/operators will provide access to the bridges and coordinate the inspections with train traffic.

2. Initial Screening Analysis. Following the collection of relevant engineering data during the inspection stage, a screening analysis would be performed to establish candidate bridges for a more detailed analysis and rating. Particular railroad bridge types were designed for high loading due to mining loads and other heavy bulk-commodity loading. An experienced engineer is capable of performing this initial screening to recognize those bridges that would not need a detailed evaluation. This stage reduces the population of bridges requiring further evaluation.
3. Detailed Structural Analysis. After an initial evaluation, a more detailed structural analysis must be conducted to determine bridge capacity to support heavy car loads under consideration. This stage will establish the candidate bridges for reinforcing scheme design and retrofit.
4. Detailed Reinforcing Design. Candidate bridges identified in stage three above will require reinforcing where bridge members and assemblies are not adequate to support the new, heavier loads. A detailed structural engineering evaluation will determine the most cost-efficient reinforcing design to increase the load-carrying capacity of the bridges. This stage will include the production of detailed calculations for each under-capacity bridge and a set of design/bid drawings and specifications for each candidate bridge.
5. Reinforcing Construction Costs. The final stage is the actual cost of the structural reinforcing construction. This stage includes the cost of the material, labor, overhead, and equipment used to strengthen members and assemblies of the bridge to meet structural requirements for heavier loads.

The described stages will form the framework for the determination of the costs to upgrade the study bridge sample. The following is a stage-by-stage evaluation of the bridge sample relative to each of the above-defined five stages.

5.1.2 Development Of Costs And Extrapolation Technique

The costs for each stage have been carefully defined to ensure an accurate statewide cost. This section provides the breakdown of the anticipated costs within each stage described above. The anticipated costs were estimated by using the experience gained from evaluating the bridge sample to increase the accuracy of upgrade cost for the population. An extrapolation procedure was then employed to determine the bridge population upgrade costs from the bridge sample upgrade costs.

5.1.2.1 Inspection

Inspection costs involve typical bridge inspection activities, which have been separated into two major categories: (1) the inspection work itself (i.e., the labor fees at each bridge) and (2) the inspection team per diem rate. Every sample bridge required an inspection to determine the relevant dimensions and conditions to assess its capacity. It was therefore anticipated that this procedure would be consistent with a statewide evaluation of short-line railroad bridges. The amount of inspection time required at each bridge was determined on the basis of experience gained from inspecting the bridges in the sample. The determination of inspection fees was determined by a review of standard fees charged by engineering firms that perform such inspections. Per diem costs were estimated due to the remote nature of many bridges in the population and is consistent with that experienced with the bridge sample.

5.1.2.2 Screening Analysis

Professional engineering costs for initial engineering review will include a cursory engineering analysis of every bridge in the population, as is consistent with the evaluation of all bridges in the sample. The engineering time required for the initial evaluation of each bridge has again been estimated on the basis of experience gained through the evaluation of the bridge sample. The engineering costs were determined on the basis of a survey of typical engineering fees.

5.1.2.3 Detailed Engineering Analysis

Detailed engineering analysis includes professional engineering costs to perform refined analyses of each bridge judged by the previous screening analysis to be insufficient to safely support the defined heavy axle loads. The fraction of the bridge population anticipated to require a detailed engineering analysis was estimated on the basis of the number of bridges that required further analysis in the bridge sample. Seven of the 25 bridges in the bridge sample extrapolates to 329 bridges of the population that can be anticipated to require detailed engineering. These 329 anticipated deficient bridges will require professional engineering analysis to ensure a thorough structural evaluation. The professional engineering time required to complete this stage for each bridge is again determined on the basis of the bridge sample. Professional engineering fees for this work are as used in the initial analysis to calculate total costs.

5.1.2.4 Detailed Strengthening Design

Detailed engineering strengthening design will produce detailed strengthening bid documents indicating necessary upgrades required to increase capacity for support of heavy car loadings. Anticipated detailed engineering costs are separated into performance of engineering design and engineering bid document production. The engineering time is estimated based on the bridge sample to complete a strengthening design and the engineering fees discussed above to determine cost. Production of bid document costs is also evaluated with time determined on the basis of the bridge sample and costs based on a survey of engineering technician fees.

5.1.2.5 Strengthening Construction Costs

Bridge strengthening costs determined for the sample bridges have been estimated based on engineering drawings submitted to qualified contractors that perform this type and scale of work. The contractor estimates defining the bridge sample costs for this stage are extrapolated to the population using the following procedure:

Step 1: The upgrade cost per sample bridge is divided by the length of the bridge to establish an upgrade cost per linear foot.

Step 2: An average unit upgrade cost per linear foot per sample bridge type is determined.

Step 3: The average unit upgrade cost per linear foot per sample bridge type is multiplied by the total length of the bridges in the population bridge type anticipated to require strengthening. This determines the anticipated upgrade cost per stratum.

Step 4: The anticipated upgrade costs per stratum are summed to determine the total bridge population upgrade cost for the short-line railroad bridges in Pennsylvania.

5.1.3 Stage Upgrade Costs

With the previous sections as background for the determination of the statewide upgrade costs for the short-line railroad bridge population, the following section details the estimated costs for each stage during the upgrade. Each section presents the assumptions and estimated fees used in the determination of the total statewide bridge upgrade cost.

5.1.3.1 Inspection Stage Costs

Average bridge inspection time (person-hours) for one bridge in each of the study strata was determined on the basis of time required to inspect the sample bridges with adjustments for experience, geography, and other factors. Additional inspection time was considered for multiple-span bridges, again based on experienced gained during the present study. Following the determination of the average person-hour inspection time per bridge type, the total inspection time per stratum was determined assuming a two-person crew. The number of bridges per stratum is shown in Table 3.2. Based on survey information, the typical engineering inspector rate at the present time was found to be \$43 per person-hour. Per diem costs for a two-person crew considered the usual lodging, food, and transportation. Based on the study sample inspection and interviews with railroad bridge inspection crews, an average of three bridges can be thoroughly inspected per day. The average per diem per bridge was determined to be \$100. Table 5.1 presents the calculation of the per diem cost per bridge to determine the per diem costs per stratum.

Table 5.1 Cost Summary for the Inspection Stage.

Stratum (Bridge Type)	Inspection Time/Bridge (hr)	Insp. Time (hr)	Inspection Time Cost	Per Diem	Total
Deck Plate Girder	3	2,802	\$120,486	\$46,700	\$167,186
Through Plate Girder	3.5	980	\$42,140	\$14,000	\$56,140
Deck Truss	4	272	\$11,696	\$3,400	\$15,096
Through Truss	4	288	\$12,384	\$3,600	\$15984
Stringer	2.5	630	\$27,090	\$12,600	\$39,690
Railtop	1.5	30	\$1,290	\$1,000	\$2,290
Masonry Arch	3	546	\$23,478	\$9,100	\$32,578
Concrete Arch	3	588	\$25,284	\$9,800	\$35,084
Concrete Stringer	2.5	35	\$1,505	\$700	\$2,205
Concrete Slab	1.5	387	\$16,641	\$12,900	\$29,541
Timber Stringer	2.5	180	\$7,740	\$3,600	\$11,340
	Total	**6,738**	**\$289,734**	**\$117,400**	**\$407,134**

5.1.3.2 Screening Analysis Stage Costs

The professional engineering time, and therefore the cost, to initially evaluate a bridge structure for ability to safely support the proposed heavy axle loads was determined for each bridge type or stratum. Using initial engineering evaluation time required for the study bridge sample, an average time was determined for each bridge type as shown in Table 5.2. This engineering time is based on the amount of time required to determine the member sizes and properties, adequately rate the superstructure, and detail the deck capacity. The average engineering time per bridge is multiplied by the number of bridges in each stratum to determine the engineering time per stratum. Based on rate survey data for experienced professional bridge engineering time, an engineering fee of $100 per hour, the stratum cost was determined.

Table 5.2 Cost Summary for the Initial Engineering Stage.

Stratum (Bridge Type)	Avg Engineering Time Per Bridge (hr)	Total Engineering. Time Per Stratum (hr)	Initial Engineering Per Stratum
Deck Plate Girder	3	1,401	$140,100
Through Plate Girder	4	560	$56,000
Deck Truss	6	204	$20,400
Through Truss	6	216	$21,600
Stringer	2	252	$25,200
Railtop	2	20	$2,000
Masonry Arch	3	273	$27,300
Concrete Arch	3	294	$29,400
Concrete Stringer	2	14	$1,400
Concrete Slab	2	258	$25,800
Timber Stringer	2	72	$7,200
	Total	**3,564**	**$356,400**

5.1.3.3 Detailed Engineering Analysis Stage Costs

Detailed professional engineering is required for 329 bridges. In previous stages, bridge upgrade costs were separated by stratum; however, for this stage bridges are considered as one group with an average engineering time applied to every bridge. Using the sample bridge times as a basis, the average engineering time for each bridge is four hours. Using this time and the engineering fee from the previous stage, the total cost for this stage is $131,600.

5.1.3.4 Detailed Strengthening Design Stage Costs

The engineering costs to complete detailed strengthening designs are calculated for the engineering time to design a strengthening scheme and time for technicians to complete the bid drawings. Based on the study bridge sample, it is expected that 235 bridges (5 of 25 sample bridges times 1,174 bridges in the population) from the total short-line bridge population will require a detailed strengthening scheme design. Based on the sample bridge detailed strengthening scheme design, it is estimated that three hours of engineering time per bridge are required, resulting in a cost for the 235 bridges of $70,500. The estimated technician time is seven hours to prepare and revise bid drawings and specifications, resulting in technician costs of $70,700 for a total of $141,200 for this stage.

5.1.3.5 Strength Construction Stage Costs

Based on detailed upgrade documents, three competent contractors provided cost estimates for each of the upgrade bridge designs. No provision was made for potential lead abatement work, which may impact the estimates. It was anticipated that railroad flagperson staffing would be accomplished with in-house personnel. Table 5.3 displays the estimate received from the three different contractors.

Table 5.3 Contractor Estimate for Bridge Strengthening.

Bridge	**Contractor**				**Average Cost per Bridge**
	A	**B**	**C**	**D**	
4A	$5,900	N/A	N/A	$11,025	$8,463
4B	$5,800	$3,900	$3,500	N/A	$4,400
8	$6,000	$4,200	$4,000	N/A	$4,733
18	$3,800	$7,900	$5,800	N/A	$5,833
19A	$8,200	$5,450	$5,800	N/A	$6,483
19B	$8,500	$5,500	$5,200	N/A	$6,400
24	$20,000	N/A	$14,600	$17,950	$17,517
				Total	$53,829

The average upgrade cost per bridge in Table 5.3 was used to perform the extrapolation procedure discussed in section 5.1.2.5 to determine the average strengthening cost per stratum shown in Table 5.4. Using these average costs, the total cost for the strengthening construction stage is calculated, as shown in Table 5.5.

Table 5.4 Average Strengthening Cost per Stratum.

Bridge No.	Average Estimate	Length (ft)	Cost per Linear Foot	Bridge Stratum	Average Cost Per Stratum
4	$4,400	45.0	$98	TPG	$162
8	$4,733	21.0	$225	TPG	
18	$5,833	16.0	$365	MAR	$365
19	$6,400	9.0	$711	TST	$711
24	$17,517	150.5	$116	TTR	$116

Table 5.5 Total Cost Summary for Strength Construction Stage (due to significant figures and rounding, the results may not be readily apparent in the Total Cost column).

Bridge Stratum	Average Cost per Foot per Stratum	Total Linear Feet of Bridge in Stratum	Total Cost per Stratum
DPG	$0	57,702	$0
TPG	$162	16,994	$2,745,880
TTR	$116	9,166	$1,066,849
DTR	$0	7,665	$0
SST	$0	4,626	$0
Railtop	$0	163	$0
MAR	$365	2,206	$804,225
CAR	$0	1,688	$0
CST	$0	236	$0
CSB	$0	2,836	$0
TST	$711	3,924	$2,790,400
		Total	**$7,407,354**

5.1.4 Width of Confidence Interval

Sampling studies such as the present study report confidence intervals to facilitate interpretation of results. The width of the confidence interval is based on a probability that the true result is located within the width. The probability is called the confidence interval, which is found by estimating the variance for the entire population. This section uses the process detailed in section 3.2 to calculate the width of the confidence interval.

The degree of freedom is determined using a t-distribution because the sample size was less than 30. The degree of freedom is determined from equation 3.5, which requires the standard deviation of the strengthening cost within a stratum and the number of bridges in each stratum within the population and sample. The standard deviation was calculated from the variance in the estimated costs from the experienced contractors. The degree of freedom is 3.388. Using linear interpolation between the degrees of freedom for a t-distribution, critical values are determined for the different confidence intervals. The estimator for the variance of the cost is multiplied by the critical values to determine the ½ the width of the confidence interval using equation 3.3. After dividing the width of the interval by the costs for stage 5, the percent around the cost for stage 5 was determined. The results are presented in Table 5.6.

Table 5.6 Widths of Confidence Intervals.

	Confidence		
	80%	**90%**	**95%**
±Width in Dollars	\$310,326	\$440,496	\$587,614
±Width in Percent	4.19%	5.95%	7.93%

5.1.5 Summary of the Bridge Upgrade Cost Determination

The first section developed the strengthening procedure for a statewide upgrade. The upgrade was separated into five different stages:

1. Inspection Stage
2. Screening Analysis
3. Detailed Engineering Analysis

4. Detailed Strengthening Design
5. Strengthening Construction Costs

Each stage contains well-defined tasks to increase accuracy in the estimate. The second section described the costs, fees, and times that would either be estimated or extrapolated from the sample bridges to the population. Sections 5.1.3 described the actual calculations for the costs. Using costs from the previous section, the final section described the width of the confidence interval for the strengthening construction costs. The bridge costs are summarized in Table 5.7, with the total statewide upgrade cost for the bridges at $8,445,000.

Table 5.7 Bridge Statewide Upgrade Costs.

Stage Description	Cost
Inspection Stage	$407,134
Screening Analysis	$356,400
Engineering Analysis	$131,600
Detailed Strengthening Design	$141,235
Strengthening Construction	$7,407,354
Total	$8,443,723

5.2 COST DETERMINATION FOR THE TRACK SURVEY

Unlike bridges, the track infrastructure database was analyzed based on data gathered from the participating SLRR owners and operators only and does not represent an extrapolation to a statewide upgrade cost. Therefore, the track survey was completed using analyses, evaluation, and cost determination procedure for the entire database. Although the track structure in the immediate vicinity of the sample bridges was inspected on site (appendix H), the values used for the required parameters were based on the completed track database (appendix G).

As a result of the evaluation of the 1,505-mile track database (section 4.2), 286,000-lb gross vehicle weight (GVW) cars resulted in 75 miles of under-capacity track and 315,000-lb GVW cars resulted in 288 miles of under-capacity track. The upgrade procedure is to replace the under-capacity rail sections with standard rail sections (see section 4.1.2). According to the evaluation, 132-lb AREA standard rail is capable of carrying both 286,000-lb and 315,000-lb car weights for all operation speed limits. However, based on cost studies, the 136-lb AREA rail sections are more commonly manufactured and, therefore, are a more cost-effective replacement rail in all situations of rail upgrade. As a result, the cost estimates are based on upgrading to 136-lb standard AREA rail sections.

The upgrade procedures included in the scope of this study assume an average but acceptable condition of cross ties and exclude tie replacement costs. Similarly, the special details for turnouts or switches are excluded from the scope of this study. Four cost estimates were gathered for the determination of an average value for the task, as listed in Table 5.8.

Table 5.8a. 286,000-lb Car Track Rehabilitation Cost Estimates.

Estimate No.	**Source**	**Unit Cost**	**Quantity (Miles)**	**Total Cost**
1	A	$211,204	75	$15,840,372
2	B	$200,000	75	$15,000,000
3	C	$254,981	75	$19,123,642
4	D	$296,481	75	$22,236,112
		Average:		**$16,654,671**

Table 5.8b. 315,000-lb Car Track Rehabilitation Cost Estimates.

Estimate No.	**Source**	**Unit Cost**	**Quantity (Miles)**	**Total Cost**
1	A	$211,204	288	$60,837,588
2	B	$200,000	288	$57,610,000
3	C	$254,981	288	$73,447,536
4	D	$296,481	288	$85,401,496
		Average:		**$63,965,040**

Detailed spread sheets for these cost estimates can be found in appendix J.

6. SUMMARY AND CONCLUSIONS

6.1 SUMMARY

The current Class I railroad trend is to begin requiring short-line railroads to accept heavy-axle cars beyond the previous standard 263,000-lb gross weight car. Gross car weights of 315,000 lb and 286,000 lb are the commonly presented weights by the Class I railroads. Given the nature of the short-line railroad infrastructure in Pennsylvania, these heavier cars are expected to be met by under-capacity of both track structures and bridge structures. The economics of short-line railroads are such that it is important that the Class I railroads be able to meet this new demand. Therefore, the present study has been undertaken to begin the statewide assessment of the short-line railroad ability to support heavier loads. In the economic interest of the Commonwealth of Pennsylvania, the PENNDOT Bureau of Rail Freight, Ports, and Waterways has funded the present study to estimate the cost for a statewide upgrade of the short-line infrastructure to accommodate the 315,000-lb and 286,000-lb gross car weights. This project investigated the infrastructure of the short-line railroads to safely support 315,000-lb and 286,000-lb gross car weights through a bridge statistical sampling scheme and a track survey.

The first phase of the present project investigated car and locomotive loading models utilized during original design as well as material properties and design specifications that were used. All short-line railroads were requested to furnish available information describing bridges and track structures under their jurisdictions. After receiving short-line infrastructure information, a sortable database was created that includes details of the bridges and track structures. Some short-line railroads do not haul freight, which resulted in their exclusion from the database. Bridges were then classified by length, construction material, and structure type.

A custom sampling procedure was developed to establish a bridge sample that accurately represents the population of short-line bridges. The bridges were divided into groups called strata according to type: deck or through plate girders, trusses, and arches. In addition, the length of each bridge was included as a substratum because the length of the bridge is a significant factor in determining upgrade cost. The number of bridges per stratum was chosen based on the procedure described in chapter 3, with a total sample size of 25 bridges.

Each bridge in the bridge sample was inspected and the results were used to determine bridge capacity. Each bridge was evaluated for force effects from five different vehicle loadings: 263,000-lb GCW, 286,000-lb GCW, 315,000-lb GCW, Cooper E-loading, and alternate live loading. Twelve of the 25 sample bridges rated at an E80 live load. Eight out of the 13 remaining bridges will safely support each of the car loadings. Five bridges were found to be under-capacity to support either the 286,000-lb or the 315,000-lb GCW, which are the focus of the present study.

Based on the above-described structural analysis, strengthening schemes were developed for each of the five bridges that could not support either the 286,000-lb or the 315,000-lb GCW. Due to the marginal nature of the loading and existing strength of the five under-capacity bridges, the strengthening schemes for both the 286,000-lb and 315,000-lb GCW load are identical, resulting in an identical statewide bridge upgrade cost for both proposals. Estimates for each strengthening scheme were obtained and upgrade costs for the bridge sample were then extrapolated to the entire bridge population. The costs were separated into stages and each stage contains tasks to aid in defining the costs accurately. The final costs for each stage are shown in chapter 5 with a total bridge statewide upgrade cost of $8,440,000. The width of the confidence interval is 7.93% for a confidence interval of 95%.

The track survey and evaluation are based on AREA 1996 Specifications as discussed in chapter 4. Based on the structural analyses of 1,505 track miles provided by participating SLRRs, 75 miles of track were found to be under-capacity for 286,000-lb rail cars and 288 miles of track were found to be under-capacity for 315,000-lb rail cars. Strengthening of under-capacity track miles will require replacement of rails with larger, 136-lb rail. Four estimates of the cost of rail replacement were obtained, with costs ranging from $200,000 to $300,000 per mile. Taking the average of the four estimates, the total cost for 75 miles of track (286,000-lb car) is $17,000,000 and for 288 miles (315,000-lb car) is $64,000,000. These costs are for upgrade of the track provided by the participating SLRRs and do not necessarily represent a statewide cost.

6.2 CONCLUSIONS

A methodology has been developed to assess the impact of heavy car GVW on the short-line railroad infrastructure across the Commonwealth of Pennsylvania. The methodology was implemented by the present study to determine the cost of strengthening the Pennsylvania short-line infrastructure to safely accommodate the proposed heavier cars. Key observations of the study are as follows:

Significant time was required for the short-line bridge and rail structure data collection. The process involved communicating with each short-line railroad and requesting sometimes untabulated information.

Sampling procedures must be performed with unbiased information and the sample chosen through a random sampling process. Stratified random sampling for the bridges was shown to best represent the bridge population and allowed the most accurate estimates of cost for bridge strengthening.

The heavy car configuration (axle spacing) to be used for the 315,000-lb and 286,000-lb car must reflect the actual configurations anticipated on the short lines. The research team investigated the anticipated commodities to be shipped in the heavier cars on the short-line railroads. The car loadings were determined from Class I railroad standards and meet specifications for coal cars, which was the heaviest commodity in Pennsylvania.

The determination of bridge member material properties without material tests is often difficult. Concrete strength varies with time and the original strength is not available. Timber bridge member species and condition is difficult to accurately assess. Steel materials were from the late 1800s to the early 1900s and again are difficult to determine, though not to the extent of other materials.

A statistical evaluation of short-line bridges and a survey of all track has resulted in a thorough evaluation of the infrastructure and its capacity to safely support heavier car weights. An estimate with confidence intervals has been determined of the costs to upgrade the Commonwealth's infrastructure to support these heavier car weights.