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16.Abstract

This study included the development of a methodology to assess the economic impact of overweight permitted vehicles hauling timber, lignite coal, and coke fuel on Louisiana highways and bridges. Researchers identified the highway routes and bridges being used to haul these commodities and statistically chosen samples to use in the analysis. Approximately 1,400 control sections on Louisiana highways carry timber, 4 control sections carry lignite coal, and approximately 2,800 bridges are involved in the transport of these commodities. Three different gross vehicle weight (GVW) scenarios were selected for this study including: 80,000 lb., 86,600 lb. or 88,000 lb., and 100,000 lb. The current GVW is 80,000 lb., the 86,600 lb. GVW is the permitted load for log trucks and the 88,000 lb. GVW is permitted for lignite coal and coke haulers. The 100,000 lb. GVW for sugarcane haulers is the highest level currently permitted by the state of Louisiana.

The methodology for analyzing the effect of these loads on pavements was taken from the 1986 AASHTO design guide and involves determining the overlay thickness required to carry traffic from each GVW scenario for the overlay design period. Differences in the life of an overlay were calculated for different GVW scenarios and overlay thickness and costs were determined for a 20-year analysis period. These costs were developed for the sample on all control sections included in the study. These present net worth costs were expanded to represent the cost for all control sections carrying each commodity.

A suggestion from enforcement personnel caused project staff to perform an additional analysis using one load axle at 48,000 lb. (48-kips), which is the maximum permissible tandem axle load. This analysis showed that 48-kip axles produce more pavement damage than the current permitted GVW for timber trucks and causes significant bridge damage at all GVW scenarios included in the study.

The methodology for analyzing the bridge costs was developed by 1) determining the shear, moment and deflection induced on each bridge type and span, and 2) developing a cost to repair fatigue damage for each vehicle passage with maximum tandem load of 48,000 lb.

Results indicate that permit fees paid by timber trucks should increase from the current $10 per year to around $346/year/truck for a GVW of 86,600 lb. when axles are equally loaded and $4,377/year/truck if 48-kip axle load are permitted. The current permit fee for lignite coal should remain at current levels. The legislature should not consider raising the GVW level to 100,000 lb. because the pavement overlay costs double over those at 86,600 lb. GVW and the bridge repair costs become significant. In many cases, the bridge costs per passage of a loaded truck amount to $8.90 meaning that the cost of bridge damage per truck per year can easily exceed $3,560.

The project staff recommends that the legislature eliminate the 48-kip maximum individual axle load and leave GVWs at the current level, but increase the permit fees to sufficiently cover the additional pavement costs produced by the presence of these permitted overweight vehicles.

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Effects of Hauling Timber, Lignite Coal, and Coke Fuel on Louisiana Highways and Bridges

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ABSTRACT

This study included the development of a methodology to assess the economic impact of overweight permitted vehicles hauling timber, lignite coal, and coke fuel on Louisiana highways and bridges. Researchers identified the highway routes and bridges being used to haul these commodities and statistically chosen samples to use in the analysis. Approximately 1,400 control sections on Louisiana highways carry timber, 4 control sections carry lignite coal, and approximately 2,800 bridges are involved in the transport of these commodities. Three different gross vehicle weight (GVW) scenarios were selected for this study including: 80,000 lb., 86,600 lb. or 88,000 lb., and 100,000 lb. The current GVW is 80,000 lb., the 86,600 lb. GVW is the permitted load for log trucks, and the 88,000 lb. GVW is permitted for lignite coal and coke haulers. The 100,000 lb. GVW for sugarcane haulers is the highest level currently permitted by the state of Louisiana.

The methodology for analyzing the effect of these loads on pavements was taken from the 1986 AASHTO design guide and involves determining the overlay thickness required to carry traffic from each GVW scenario for the overlay design period. Differences in the life of an overlay were calculated for different GVW scenarios, and overlay thickness and costs were determined for a 20-year analysis period. These costs were developed for the sample on all control sections included in the study. These present net worth costs were expanded to represent the cost for all control sections carrying each commodity.

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The methodology for analyzing the bridge costs was developed by 1) determining the shear, moment, and deflection induced on each bridge type and span, and 2) developing a cost to repair fatigue damage for each vehicle passage with maximum tandem load of 48,000 lb.

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The project staff recommends that the legislature eliminate the 48-kip maximum individual axle load and leave GVWs at the current level, but increase the permit fees to sufficiently cover the additional pavement costs produced by the presence of these permitted overweight vehicles.

ACKNOWLEDGMENTS

This report could not have been completed without the assistance of personnel from Districts 03, 04, 05, 07, 08, 58, 61, and 62. Personnel from district administration, construction engineering, maintenance, materials, and traffic all contributed to the successful completion of the project. Each district provided personnel to meet with project investigators to estimate the pavement cross sections for each control section in the district carrying timber. They later developed the history of pavement construction and rehabilitation, and then made traffic volume and classification counts on each control section included in the study. Without this timely assistance, we simply could not have performed the study.

In addition to LADOTD personnel, representatives of the Louisiana Forestry Association developed estimates of the tonnage of timber that was hauled over each of the control sections included in the study. The authors especially want to thank Buck Vanderstein, executive director of the forestry association, for his help in coordinating the collection of this information. Representatives from Savage Industry were also very helpful in determining the amount of lignite coal hauled, the route trucks followed, and the truck configuration and weight distribution on the axles of those trucks.

Lastly, we want to express our gratitude to the Project Review Committee, many of whom provided direct assistance to the project team as we developed information needed to complete the study.

Members of the Project Review Committee included:

Arthur D. Johnson, Savage Industry
Randy Sines, Savage Industry
Larry Creasy, Department of Public Safety
Wallace Davidson, Department of Public Safety
John Wells, LADOTD
Said Ismail, LADOTD
Michael Boudreaux, LTRC
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Billy Metcalf, LADOTD
Masood Rasoulian, LTRC
Buck Vanderstein, La. Forestry Association
Phil Arena, Federal Highway Administration

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IMPLEMENTATION STATEMENT

The results from this project can be immediately implemented by the Louisiana legislature in the 2005 legislative session. Study results show that the most significant statute change needing repealing is the 48,000 lb. maximum individual tandem axle load limit. This provision alone induces annual pavement and bridge damage of over $40 million, with most of the damage to bridges. None of the cost of this damage is currently being recovered through permit fees. As a result, this $40 million annual cost represents a direct subsidy to the timber industry. A review of the pavement and bridge costs could compel the legislature to define the level of subsidy provided to the timber, lignite coal, and coke fuel industries. In analyzing the effect of the current GVW defined by Louisiana statutes, the project staff determined that at the current 86,600 lb. GVW prescribed for timber trucks, the legislature provides a minimum pavement damage subsidy of $346 per vehicle per year for equally loaded axles. This minimum value is based on the assumption that all agricultural harvest permits are log trucks, which is clearly not accurate. The bridge study indicates that the effect of log trucks with individual axles loaded to 48,000 lb. produces a minimum cost of $3,560/year/truck. Therefore, the project staff recommend:

Eliminating the 48,000 lb. maximum individual tandem axle load limit
Requiring equal loading on both the truck and trailer axles
Increasing the permit fee for 86,600 lb. GVW harvest permits from $10 to $346/year/truck.

However, when investigating the effect of increasing the GVW to 100,000 lb., the added cost of overlays increased by $385/year/truck when compared to current conditions. Bridge repairs increased from zero to $8.90 for each passage of a log truck loaded with maximum tandem load of 48,000 lb. for an estimated annual cost of damage of $3,560/truck. As a result, the project staff recommends that no consideration be given to increasing the GVW from current levels to 100,000 lb.

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INTRODUCTION

During the 2004 regular session, the Louisiana senate passed a concurrent resolution (Senate Concurrent Resolution 123), sponsored by Senators Smith and McPherson, which urged the Louisiana Department of Transportation and Development (DOTD) to study the laws governing the operation of vehicles that haul Louisiana products in excess of the standard limitations set forth in law. Resolution 123 specifically requested that the study include vehicles hauling timber, lignite coal, and coke fuel. In addition, the resolution asked the DOTD to study the laws that govern operation of all vehicles that haul Louisiana products in excess of the standard limitations set forth in law to make recommendations to the legislature and offer proposals for legislation to update such laws. Resolution 123 also requested that the following issues be included in the study:

1. The economic impact to the state and to the industry should loads be permitted to exceed the present legal limitation set forth in law,
2. The fiscal impact on the state should loads be permitted to exceed the present legal limitation set forth in law,
3. The adequacy of current special permit fees assessed on trucks, semi-trailers, truck-tractors, tandem trucks, or combinations, and other similar vehicles, both when in compliance with standard limitations and when in excess of standard limitations, which operate on Louisiana's highways, roadways, and bridges, and
4. A review of the surrounding states' laws that govern the operation of heavily loaded vehicles on highways, roadways, and bridges.

The purpose of this report is to describe the work conducted to address the issues raised by Senate Concurrent Resolution 123. The following is a description of current situation for gross vehicle limits for timber, lignite coal, and coke fuel.

Current state laws allow truck operators hauling certain agricultural and natural resource commodities to purchase overweight permits and haul at gross vehicle weights (GVW) that exceed the legislated limit of 80,000 lb. Among these agricultural and natural resource commodities is timber, which is harvested in all but two Louisiana parishes. However, the industry is concentrated in the northern and central portions of Louisiana and the Florida parishes. Table 1 contains the dollar value of forestry products harvested in Louisiana parishes in 2003. The vast majority of these totals are timber products. As shown in Table 2, forestry products accounted for almost 22 percent of the total agricultural production in Louisiana in 2003. Since forestry is such an important part of Louisiana's economic base, any changes in the legal weight or overweight permit structure for Louisiana must consider the potential impact on the forestry product industry as well as the cost to maintain and rehabilitate the roads and bridges used by vehicles hauling forest products.

The only two lignite coal mines in Louisiana are the Dolet Hills mine near Mansfield and the Oxbow mine near Armistead. All of the lignite from both mines is used to power the Dolet Hills power plant. The lignite mined from the Dolet Hills mine leaves the mine area by off-road truck and is carried to the crusher. The crushed lignite then travels on a 7.5-mile conveyor to the power plant, never leaving mine property. The lignite produced at the Oxbow mine travels from the mine on La 177 to US 84 to LA 3248 to the power plant, according to personnel of Red River Mining Co., which operates the mine. Personnel of Red River Mining indicated that all of the 750,000 tons/year of lignite produced at the Oxbow mine goes to supply the energy needs of the Dolet Hills power plant.

When petroleum refineries process petroleum without producing asphalt, the residuum is cracked to reduce the heavy products and produce lighter constituents. The bottom of the barrel product, coke, is used for fuel to produce electricity and to power ocean going vessels. Coke is also calcined for use in aluminum production and used in steel production, among other uses. This coke is transported to the end users by rail cars, ocean going vessels, barges, and trucks. This project addressed only the trucks that transport coke fuel.

Since transport vehicles hauling these products can purchase overweight permits to carry loads in excess of the 80,000 lb. GVW, this study will evaluate the highway cost consequences created by permitted vehicles hauling these commodities. Highway costs will be generated for three scenarios:

Scenario 1-vehicles hauling each commodity at 80,000 lb. GVW.
Scenario 2-vehicles hauling at the currently permitted load.
Scenario 3-vehicles hauling each commodity at 100,000 lb. GVW.

Bridge costs will be generated for maximum tandem load of 48,000 lb., which corresponds with Scenario 3 loads plus the load factors included in the Load Resistance Factor Design (LRFD) method of design.

Table 1
2003 forestry harvest value for Louisiana parishes

Parish	2003 Value, million $	Parish	2003 Value, million $
Acadia	6.220	Madison	2.706
Allen	42.451	Morehouse	19.161
Ascension	0.714	Natchitoches	32.566
Assumption	0.075	Orleans	0.009
Avoyelles	5.660	Ouachita	10.775
Beauregard	61.137	Plaquemines	0.003
Bienville	61.675	Point Coupee	5.200
Bossier	24.082	Rapides	37.600
Caddo	18.375	Red River	12.409
Calcasieu	9.599	Richland	1.755
Caldwell	13.059	Sabine	60.160
Cameron	0.000	St. Bernard	0.000
Catahoula	8.456	St. Charles	0.445
Claiborne	46.410	St. Helena	11.144
Concordia	3.542	St. James	0.231
Desoto	34.826	St. John	0.070
East Baton Rouge	3.618	St. Landry	4.179
East Carroll	2.320	St. Martin	0.569
East Feliciana	11.670	St. Mary	0.009
Evangeline	14.461	St. Tammany	9.471
Franklin	0.916	Tangipahoa	17.284
Grant	13.057	Tensas	2.511
Iberia	0.011	Terrebonne	0.087
Iberville	1.850	Union	43.491
Jackson	48.798	Vermillion	0.086
Jefferson	0.188	Vernon	67.576
Jefferson Davis	1.940	Washington	27.961
Lafayette	0.245	Webster	32.766
Lafourche	0.022	West Baton Rouge	0.588
Lasalle	22.680	West Carroll	0.757
Lincoln	16.848	West Feliciana	6.063
Livingston	26.392	Winn	47.114

Table 2
Total value of agriculture in Louisiana for 2003

Crop	Gross Farm Value in 2003, million $
Cotton	311.491
Forestry	956.352
Fruits	17.835
Feed Grains	194.062
Greenhouse Vegetables	1.726
Hay for Sale	36.349
Home Gardens	118.463
Nursery Crops	106.974
Pecans	15.069
Rice	152.098
Sod Production	14.875
Soybeans	182.521
Sugarcane	359.020
Sweet Potatoes	87.653
Vegetables	39.906
Wheat	19.527
PLANT ENTERPRISES TOTAL	2,614.137
Fisheries & Wildlife Enterprises TOTAL	409.539
Animal Enterprises TOTAL	1,331.090
All Agricultural Enterprises TOTAL	4,354.765

OBJECTIVES

The principal objectives of this study were to:

1. Assess the impact of vehicles hauling forestry, lignite coal, and coke fuel products on the maintenance and rehabilitation of Louisiana state highways and bridges under current Louisiana laws that set forth gross vehicle weights, and assess the permit structure that describes the conditions under which legal overweight permits may be purchased.
2. Provide the legislature with proposal that would modify current laws by providing new weight restrictions to reduce damage to Louisiana state highways and bridges, while keeping these Louisiana industries economically viable.

SCOPE

The primary thrust of this project was to assess the magnitude of highway and bridge rehabilitation costs incurred by vehicles hauling timber, lignite coal, and coke fuel on Louisiana highways and bridges in excess of the 80,000 lb. gross vehicle limit and hauling with one axle loaded to the 48,000 lb. individual axle load. Some trucks hauling timber start out on parish roads adjacent to the land where the timber is harvested. However, this study concentrated on determining costs for highways and bridges that the DOTD is responsible for constructing, rehabilitating, and maintaining. In addition, off-system bridge inventory data was reviewed to develop a preliminary order of magnitude indication of the effect of increased axle loads and gross vehicle weights on the performance of these bridges. Highways used to haul lignite coal and coke fuel were also identified and the effect of transporting these commodities on highway cost was determined.

This study was begun in July 2004 as a direct result of Louisiana Senate Concurrent Resolution 123 which required that the DOTD prepare and submit a report back to the senate in March 2005. To conduct such a study would normally take two years. Because of the time constraint, some assumptions and limitations were required to complete the work. Among these limitations are:

1. The make up of the pavement structure and the history of construction for each control section. Many control sections had no readily available information and best estimates were made by the most knowledgeable district personnel.
2. The subgrade soil modulus values used were averages for each parish. These averages were almost certainly too large for many of the control sections. The effect was to make the calculated overlay thickness and cost smaller than would likely be the case if there was time to develop more detailed information.
3. The m-values included in the structural number equation were assumed to be 1.0 since most of these control sections were designed under pre-1986 design procedures. Using the 1986 flexible pavement design guide and appropriate values for m would produce increased thickness and cost for many overlays.
4. Traffic volumes included in the latest control section books may be inaccurate. If these ADT values are too low, the calculated truck axle loads for design of the overlays will be too low and the calculated overlay thickness and cost will be low.
5. Estimates of timber tonnage hauled on each of the 39 control sections included in the study were based on estimates of knowledgeable industry personnel and not from actual data taken from mill records. The accuracy of the data developed by the timber industry is consistent with the level of accuracy of much of the data on the pavement cross sections and ADT data.
6. The fatigue cost for bridges was determined based on the average cost for projects completed by LADOTD in 2004.

METHODOLOGY

The first item of work was to define the Louisiana highways and bridges on which timber, lignite coal, and coke fuel are hauled. The project staff was very successful in identifying the roads used to transport timber and lignite coal. However, little information could be found on either the quantity of coke fuel transported or the routes used to haul it. In the following sections, the routes involved in transporting each commodity will be identified and the sample of roads and bridges included in the study described. Then the calculation methodology developed to estimate the overlays required to support transportation of the commodities under the various gross vehicle weight scenarios will be described. Lastly, the evaluation methodology for bridge effects will be described.

Identifying Control Sections Carrying Timber

To identify state roads used for transporting timber products, project staff worked with the Louisiana Forestry Association (LFA), which is headquartered in Alexandria. Using DOTD district maps, the staff of the LFA identified the roads currently used to haul timber products. Project staff then used the DOTD control section books and maps to list all the control sections in each district carrying timber traffic. A summary of the number of control sections by district is shown in Table 3.

Table 3
Control sections carrying timber products by La DOTD district

District No. & Office Location	No. of Control Sections carrying Timber Products
2	0
3	120
4	291
5	193
7	66
8	299
58	88
61	175
62	180
TOTAL	**1,412**

Once the control sections were identified, project staff visited each district office to meet with district personnel including maintenance engineers, maintenance specialists, design engineers, and construction engineers to generate the typical pavement cross section for each control section. While many control sections had more than one cross section, project staff made it clear that the predominant cross section along the control section was needed. As a result, project staff were able to record this information in most districts during a one-day meeting. Information collected included the layer type and thickness for surfaces and bases for each control section. Project staff told district personnel that accuracy to within 1 to 2 inches for individual layers was sufficient for this study. As a result, locating construction plans for control sections was not necessary; indeed, the six-month duration of this study did not allow time for that kind of detail. As a result, most of the layer thickness information was developed from the field experience of the district personnel in working on different control sections.

Once the pavement cross sections were defined, the structural number was calculated for each control section. The structural number (SN) is defined as the sum of the relative strength coefficient, a, times the layer thickness, D, for all layers in the pavement:

$$SN = a_1 * D_1 + a_2 * D_2 \quad (1)$$

Where,

a_1 = relative strength coefficient for the wearing & binder courses
D_1 = combined thickness of the wearing & binder courses
a_2 = relative strength coefficient for the base course
D_2 = thickness of the base course

NOTE: the m term included in the 1986 AASHTO flexible pavement design guide has been omitted from the calculation of structural number because most of the pavements in this study were designed before that guide took effect.

To calculate a weighted structural number for each control section, the SN was multiplied by the length of the control section, in miles, by the number of lanes. Once the ADT (average daily traffic) sample groupings were defined, the weighted structural number for all control sections in each group was summed and then divided by the sum of the length times the number of lanes to get the average structural number for that group. The standard deviation of the structural number was also calculated for each ADT group.

Sample Size Calculations for Control Sections Carrying Timber Products

The sample size for each ADT group was determined using the central limit theorem of statistics [1]:

$$n = \{[(Z_{alpha/2}) * Sigma] / \{[(\% \text{ error in the estimated mean})/100] * M\}\}^{1/2} \quad (2)$$

Where,

n = the size of the sample needed to give an acceptable estimate of the mean
$Z_{alpha/2}$ = Value of the standard normal deviate at an error level of alpha/2
alpha = magnitude of the type 1 error willing to be tolerated
Sigma = the standard deviation of all observations in the data set % error in the estimated mean = the error in the estimated mean, for example, if the estimate of the mean is to be within 10% of the actual mean, then the % error

is 0.10 * M
M = the mean or average of all observations in the data set

To make these calculations, values had to be selected for alpha and percent error in the estimated mean. These values were chosen with two things in mind: the accuracy of the results of the study and the time available to perform the study. In selecting an accuracy level to estimate the sample size, project staff were significantly influenced by the ability of district personnel to provide typical pavement layer thickness data. Since the estimated thickness of both the hot mix asphalt surfacing and base materials was within two inches, the SN was between 0.28 and 0.88 (for the base: SN = 0.14 * 2 = 0.28, and for the wearing course: SN = 0.44 * 2 = 0.88). To determine the percent of the mean that these numbers represents, the mean (M) for each ADT grouping was calculated. Each group is identified in table 4 along with the mean of the SN, the standard deviation of SN for each group, and the calculated number of control sections needed to estimate the effects of timber trucks within 20 percent of the mean SN 90 percent of the time.

Table 4
ADT grouping of control sections along with mean, standard deviation of structural number (SN), and required sample size

ADT Range	# of Control Sections	Calculated Mean of SN	Calculated Standard Deviation of SN	No. of control sections required
Less than 1000	504	2.224	0.973	13
1000 to 4000	497	3.319	1.521	15
Greater than 4000	411	4.918	1.899	11
TOTALS	**1412**			**39**

After the number of control sections needed for each ADT group was determined, the control sections to be included in the cost analysis were selected. The selection process involved using a random number generator program secured from the Internet. The program was written in Visual Basic and defined a function "calcrandnum." The function executed the program using the "RND" syntax which generated random numbers. Three variables "upp," "low," and "r" were required as input. The number of control sections in each ADT group was "upp," and "low" was one, the number of the first control section in the range. The program asked for the number of selections to be made, that is, the sample size, which was "r." The program was then executed to produce a set of "r" random numbers. Once the random numbers were generated, the control section corresponding to each number was identified, as shown in table 5 for each of the ADT groups.

Table 6 contains a brief description of each control section included in the sample. The control sections have been arranged by DOTD district to show the distribution of control sections across the state. Each district received this list of control sections and was asked to confirm the pavement cross sections, determine when the road was built and the original cross section, determine when the last major rehabilitation was performed on each control section, and describe the last major rehabilitation. In addition, the traffic section of each district was asked to conduct field traffic surveys on all the control sections to provide an accurate ADT and a classification count for each vehicle type. The traffic section in Baton Rouge provided the traffic growth rate for each control section.

Table 5
Identification of control sections included in each ADT group

ADT Group	Random number ID for each control section	Route number	Control section number	District number
ADT < 1,000 (504 control sections in this range)	24	La 771	830-01	58
	82	La 548	323-01	5
	83	La 968	863-10	61
	141	La 591	834-08	5
	144	La 963	819-19	61
	150	La 500	128-02	8
	153	La 126/503	130-02	58
	194	La 169	45-31	4
	203	La 154	88-06	4
	208	La 151	317-05	5
	327	La 151	89-06	5
	479	La 8	134-02	8
	495	La 464	136-01	8
1000$\leq$ ADT < 4000 (497 control sections in this range)	41	La 757	849-26	3
	93	La 9	89-03	4
	104	La 1050	853-05	62
	131	La 110	190-02	7
	145	La 38	263-02	62
	163	La 1054	853-12	62
	189	La 416	224-01	61
	206	La 169	48-02	4
	229	La 115	805-18	8
	291	La 1056	853-14	62
	316	La 2	83-01	4
	355	La 63	272-02	62
	391	La 43	260-07	62
	451	La 413	227-02	61
	458	La 1062	415-04	62
ADT $\geq$ 4000 (411 control sections in this range)	10	La 482	842-09	8
	41	La 27	31-07	7
	177	La 423	817-31	61
	224	US 190	8-03	61
	232	La 64	253-04	61
	259	La 34	67-09	5
	261	US 190	12-13	3
	286	La 156	92-02	8
	344	La 67	60-01	61
	377	La 34	67-09	5
	378	La 1	53-09	4

Table 6
Control section numbers, cross sections, and ADT by DOTD district number

Dist. No.	Route No.	Control Section No.	ADT	W.C& B. C. Thick., in.	Base Type & Thickness, in.
3	La 757	849-26	1114	3.5”	8.5”, soil cement
	US 190	12-13	11639	8.0”	8”, PCC
4	La 169	45-31	400	3.5”	6”, soil cement
	La 154	88-06	420	3.5”	8.5”, soil cement
	La 9	89-03	1270	3.5”	8.5”, soil cement
	La 169	48-02	1805	3.5”	8”, PCC
	La 2	83-01	2423	3.5”	8.5”, soil cement
	La 1	53-09	25986	5.5”	8”, PCC
5	La 548	323-01	246	3.5”	8.5”, soil cement
	La 591	834-08	344	Surface Trt.	6”, sand clay gravel
	La 151	317-05	422	Surface Trt.	6”, sand clay gravel
	La 151	89-06	636	3.5”	8.5”, sand clay gravel
	La 34(2-lane)	67-09	11602	3.5”	8.5”, soil cement
	La 34(4- lane)	67-09	25182	5”	9”, PCC
7	La 110	190-02	1432	3.5”	8.5”, soil cement
	La 27	31-07	4582	6”	8.5”, soil cement
8	La 500	128-02	353	1.5”	8.5”, soil cement
	La 8	134-02	947	3.5”	8.5”, sand clay gravel
	La 464	136-01	976	3”	8.5”, soil cement
	La 115	805-18	1867	3.5”	12”, soil cement
	La 482	843-09	4154	3.5”	8.5”, soil cement
	La 156	92-02	13763	3.5”	8.5”, soil cement
58	La 126/ 503	130-02	361	3.5”	8.5”, soil cement
	La 771	830-01	142	Surface Trt.	6”, sand clay gravel
61	La 968	863-10	248	Surface Trt.	9”, sand clay gravel
	La 963	819-19	348	Surface Trt.	9”, sand clay gravel
	La 416	224-01	1721	3.5”	8.5”, soil cement
	La 413	227-02	3466	3.5”	8.5”, soil cement
	La 423	817-31	7696	3.5”	8.5”, soil cement
	US 190	8-03	9774	3.5”	8.5”, soil cement
	La 64	253-04	9933	3.5”	8.5”, soil cement
	La 67	60-01	20516	3.5”	8.5”, soil cement
62	La 1050	853-05	1305	Surface Trt.	12”, sand clay gravel
	La 38	263-02	1494	3.5”	12”, soil cement
	La 1054	853-12	1595	2”	12”, soil cement
	La 1056	853-14	2256	3.5”	12”, soil cement
	La 63	272-02	2646	12”	8.5”, soil cement
	La 43	260-07	2963	3.5”	8.5”, soil cement
	La 1062	415-04	3537	Surface Trt.	12”, sand clay gravel

Identifying Control Sections Carrying Lignite Coal

Lignite coal is produced at two mines in northwest Louisiana in Red River and Desoto parishes at the Dolet Hills and Oxbow mines. All the coal mined at the Dolet Hills mine is transported via conveyor from the mine to the power plant. The only lignite coal transported on Louisiana highways travels from the Oxbow mine at Armistead, Louisiana

a. Along La 1 for about 8 miles to the point where US 84 diverges west,
b. The coal then moves approximately 6 miles along US 84, past I49, to La 3248, and,
c. Along La 3248 for approximately 2 miles where the trucks turn onto the road to the Dolet Hills power plant.

Since all the highways carrying lignite coal are in District 04, they were asked to supply pavement cross section and history information. The traffic section in Baton Rouge was asked to supply ADT, vehicle classification data, and traffic growth rates for each of the control sections carrying lignite coal as shown in table 7. However, the traffic section in Baton Rouge indicated that no ADT data was readily available for these control sections, so the district traffic personnel collected traffic count and classification data on these four control sections. The traffic section in Baton Rouge provided estimates of traffic growth rate.

The pavement cross section data for US 84 was secured using ground penetrating radar data collected for the DOTD in 1995. That data was supplemented with information from District 04 personnel to develop the current cross section. In addition, District 04 personnel provided data on rehabilitation activities on each of the control sections included in table 7.

Table 7
Control section numbers, cross sections, and ADT for roads carrying lignite coal

District No.	Route No.	Control Section No.	ADT	W.C& B. C. Thickness, in.	Base Type & Thickness, in.
4	La 1	53-07	1608	7.0	Soil Cement, 8.5
	US 84	021-04	909	9.5	Soil Cement, 8.5
	US 84	21-03	1122	6.5	Soil Cement, 8.5
	La 3248	816-07	335	5.0	Soil Cement, 8.5

Identifying Control Sections Carrying Coke Fuel

Project staff contacted Louisiana refineries to determine how much of their coke was transported over Louisiana highways. This information will be discussed in the results section of this report.

Calculation Procedure to Estimate Highway Overlay Costs for Overweight Vehicles

The following calculation procedure was developed to study the effect of trucks carrying timber, coke fuel and lignite coal on Louisiana highways. This procedure was applied separately for each commodity. The description below applied to timber, but was also used for the transport of lignite coal and coke fuel.

1. Secure pavement design data from DOTD to have information on design of the latest major rehabilitation on each control section. Other data needed includes the pavement cross-section, date of construction, the current ADT, subgrade resilient modulus and other required data for an assessment of the effects of different GVWS on rehabilitation costs.

2. For each control section, determine how many tons of each commodity was hauled over the road during 2003 on the way from the production point to the first processing or use point. This data was developed with the assistance of industry personnel who work with each commodity.

3. Using the data, estimate the time when the existing control section will carry all the design traffic for each weight scenario. The weight scenarios investigated for the Louisiana Type 9 vehicle carrying timber are shown in table 8. Scenario 2 represents the present situation, in which the timber trucks carry 86,600 lb. GVW.

Table 8
Type 9 axle loads for vehicles carrying timber for each GVW weight scenario

GVW Scenario	Highway type	Steering Axle lbs	Tandem Axle lbs	Tandem Axle lbs	GVW, lbs.
Scenario 1	State & US	12,000	34,000	34,000	80,000
Scenario 2	State & US	12,600	37,000	37,000	86,600
Scenario 3	State & US	12,000	44,000	44,000	100,000

Table 9
Timber payload for each weight scenario

GVW Scenario	Highway Type	Sum of Axle Loads, lbs	Vehicle Empty weight, lbs	Payload/Truck, lbs
Scenario 1	State & US	80,000	26,600	53,400
Scenario 2	State & US	86,600	26,600	60,000
Scenario 3	State & US	100,000	26,600	73,400

4. For each weight scenario, determine the empty weight of type 9 trucks so that the average payload per truck can be determined (Payload = GVW – empty weight). Table 9 shows an example of payload calculation for type 9 trucks carrying timber. Calculate the number of trucks required to carry the commodity by dividing the total

weight hauled over the road by the average payload. This number of trucks was appropriately added into the traffic estimates for each scenario.

5. Once the current design traffic has been served, redesign an overlay for each roadway assuming that each weight scenario continues during the next overlay design period, which is eight years for Louisiana. Repeat this procedure for the length of the analysis period to generate a project cost stream including these periodic rehabilitations.

6. Calculate the net present worth of the rehabilitation costs for each project using an interest rate provided by the DOTD. An interest rate of five percent is suggested for these calculations.

7. Compare the cost differential for the various weight scenarios and develop cost differential tables for comparisons between the weight scenarios.

8. Estimate the statewide cost impact for the cost differentials developed in step seven.

The following paragraphs describe each step in the pavement effects methodology.

Step one involved securing pavement design data from different district offices on the most recent rehabilitation on each construction. In addition, project construction history data was provided along with ADT and vehicle classification data collected by the district on each control section. Pavement layer thickness and material type were also provided by the district. Traffic growth rates were provided by the Traffic Monitoring and the System Inventory section of the DOTD. The pavement and geotechnical design section provided sub-grade soil resilient modulus data in addition to other pavement design parameters, including initial and final serviceability levels and reliability values for each roadway type, a- and m-values used for various materials, and the overall standard deviation used to design Louisiana pavements.

Step two involved determining the quantity of each commodity hauled over a particular control section in 2003, the base year of the study. This data was provided by industry representatives. The provided data represents the total 2003 payload carried by trucks transporting each commodity over each control section. The number of trucks required to carry the total payload was calculated by dividing the total payload by the payload carried by each truck for each of the various GVW scenarios.

Step three involved defining the weight scenarios to be investigated for each commodity included in the study. The base scenario was assumed to be that in which all vehicles operate according to the legal loading with no special permits. Scenario 1 provides a basic picture of how the pavements will perform without special overweight permits for agricultural products. As anticipated in a preliminary study, the three weight scenarios for timber are 80,000 lbs., 86,600 lbs., and 100,000 lbs.

For each weight scenario, the payload per truck was determined in step four. The payload per truck was calculated by subtracting the empty weight of the truck from the sum of the axle weights for the vehicle, which are shown individually in table 8 and recorded in table 9. Using the total commodity hauled and the average payload per truck, the number of vehicle-trips required to carry the total weight of commodity could be calculated for each commodity. The average empty weight of the type 9 vehicle carrying timber was considered to be 26,600 lbs [2].

Since timber operations generally occur year-round, these vehicles are assumed to be included in the traffic projections at the current permitted level of 86,600lbs. Therefore, the number of trucks required to carry the timber under scenario 2 was included in the current pavement design traffic volume estimates. The number of timber trucks required to carry the total payload under scenario 2 will be subtracted from the type 9 traffic stream for scenarios 1 and 3, and a new number of trucks with different payloads (and axle loads) was added back in to complete the traffic estimates for scenarios 1 and 3.

Step five involved taking the pavement design traffic in ESALs, construction date of the most recent rehabilitation, and traffic growth rate to estimate how much of the design traffic has been carried by each control section at the end of 2003. The difference between the design traffic and that carried to the end of 2003 was applied using the three weight scenarios presented earlier to estimate a date when the total design traffic has been carried by the road and rehabilitation is needed. Traffic for the new rehabilitation was developed by projecting the previous traffic. Therefore, three traffic estimates were made for each control section in each rehabilitation that occurs during the analysis period. A cost stream was generated for each scenario in each control section, representing the rehabilitation costs that are included during the analysis period.

Step six involved computing the net present worth of rehabilitation costs for each control section under the three different weight scenarios. The interest rate used in these calculations was 5.0 percent.

Step seven involved developing comparisons between the three different weight scenarios. The comparisons between scenarios 1 and 2 indicated the pavement costs associated with moving from the no permit weights (scenario 1) to the current permits on each control section (scenario 2). A second comparison of special interest was that between pavement costs associated with moving from scenario 2 to scenario 3, which allows up to 100,000 lbs. on all control section for the FHWA type 9 vehicle carrying timber.

Step eight involved taking the data for the control section in each ADT group and expanding that data to produce a statewide estimate of the effect of each GVW scenario. This statewide estimate was developed by multiplying the number of miles statewide in each ADT group by the number of miles in the study control sections by the cost difference from step seven.

Example Demonstrating Use of Methodology

As discussed in the methodology, road sections transporting each commodity were identified. The pavement design data was secured for the control sections constituting these highways. This data was used along with commodity estimates transported over each control section to predict the effect of the additional ESAL on 1) the time to the next overlay and 2) the amount of overlay required. These data were then used to generate a DOTD cost stream for each weight scenario. Net present worth was calculated for each scenario and the differences between the net present worth for the GVW scenarios provided a basis for comparing the effects of the different weight scenarios on pavement costs.

To demonstrate how these calculations were performed, a pavement section on US 84 over which timber was transported is included next.

US 84 Carrying Timber. In 1998, 550,000 tons of timber was hauled on US 84 (50 percent in each east & west direction), which translates into 275,000 tons in the design lane of this 2-lane road.

$$\text{Weight of Timber transported } = 275{,}000 * 2{,}000 = 550{,}000{,}000 \text{ lbs}$$

The current GVW condition is represented by scenario 2 under which the permitted vehicles carry 86,600 lbs. Hence, scenario 2 will be discussed before the other two scenarios. Information provided by the DOTD showed that the terminal serviceability index (P_t) for this highway is 2.0. To determine the truck factor for a log truck loaded at the GVW, the structural number (SN) was assumed to be 4.0.

The 20-year analysis period included in the sample calculation is from mid-1999 to mid-2019. As a result, the overlay thickness required to carry traffic for this 20-year period was determined and the mid-1999 present net worth was calculated for each of the three GVW scenarios.

Calculation of ESAL for the first performance period under current conditions (Scenario 2). For a timber truck loaded to 86,600 lbs. GVW, the following axle configuration was used and the load equivalence factors were obtained from tables D1 and D2 of AASHTO [3] for SN = 4.0 and P_t = 2.0;

Steering Axle (12,000 lbs.) = 0.183
Tandem Axle (37,300 lbs.) = 1.601
Tandem Axle (37,300 lbs.) = 1.601
ESALs per Truck = 3.385 ESALs

Maximum Payload per truck = GVW – tare weight of truck
= 86,600 – 26,600 = 60,000lbs.

Hence the number of trucks required to carry the timber in 1998 under scenario 2 with a GVW of 86,600 lbs = $\dfrac{550{,}000{,}000}{60{,}000} = 9{,}167$ trucks/year = 25 trucks/day

For the traffic distribution and 1998 ADT, the number of 18-kip ESALs for the first performance period of scenario 2 (present conditions) was calculated as shown in table 10. It was assumed that the 1998 overlay was calculated for the traffic calculated in table 10.

Table 10
Calculation of ESALs for 1998 to 2006 under present GVW conditions (scenario2)

Timber on US 84, Rural Major Collector

Performance Period:	8	years(Overlaid section)
Average Daily Traffic in 1998:	3232	Last Overlaid in : 1998
Directional Distribution Factor:	50	%
Lane Distribution Factor:	100	%
Annual Growth of Non - Timber Traffic:	1	%/year
Growth Factor for Non-Timber Traffic:	8.2857	
Annual Growth of Timber Traffic:	0	%/year
Growth Factor for Timber Traffic:	8	

FHWA Class	%ADT	ADT Per Class	% Annual Growth	Growth factor	T.F	18k ESAL*
1	0.3	10	1	8.2857	0.0004	6
2	66.1	2136	1	8.2857	0.0004	1,292
3	21.1	682	1	8.2857	0.0143	14,746
4	0.4	13	1	8.2857	0.1694	3,312
5	1.4	45	1	8.2857	0.1694	11,591
6	3.2	103	1	8.2857	0.3836	59,992
7	0.2	6	1	8.2857	0.3836	3,749
8	1	32	1	8.2857	0.8523	41,654
9a (Non-Timber)	5.3	146	1	8.2857	1.045	230,994
9b(Carrying Timber)		25	0	8.0000	3.385	248,233
10	0.7	23	1	8.2857	1.45	49,605
11	0.1	3	1	8.2857	1.84	8,992
12	0	0	1	8.2857	1.84	-
13	0.2	6	1	8.2857	1.84	17,985
Total	100	3232				692,151

***Design lane Traffic = ∑(Col.3)X(Col.5)X(Col.6)X(365)X(0.5)X(1.0)**

Since timber operations generally occur year-round, timber trucks are assumed to be included in the traffic data collected by the districts in this study. Therefore, the number of trucks required to carry the timber under scenario 2 was included in the current pavement design traffic volume estimates. The number of type 9 trucks that do not carry timber was then calculated by subtracting the timber trucks from the total number of type 9 trucks, (see column 3 of table 10). The design lane ESAL calculations were then made for a performance period of eight years from the time since the last overlay was placed (1998 for US 84). The performance period for overlays designed by the DOT includes traffic for a period of eight years.

Calculation of ESALs for the second performance period. ESAL calculations similar to those generated for the first performance period were generated for the second performance period and are included in table 11. The traffic was projected using the traffic growth factors provided by the traffic section in Baton Rouge. The ADT for the beginning of the second performance period was generated by multiplying the 1 percent growth per year for 8 years, or 3,500 vehicles per day. The distribution of traffic was assumed to remain constant during the 20-year analysis period, i.e., the percentage of each vehicle type does not change. However the number of log trucks (Type 9 carrying timber) changed with the different GVW scenarios.

Table 11
Calculation of ESALs for 2006 to 2014 under present GVW conditions (scenario2)

Timber on US 84, Rural Major Collector

Performance Period:	8	Years(Overlaid section)
Average Daily Traffic in 2006:	3500	Last Overlaid in: 2006
Directional Distribution Factor:	50	%
Lane Distribution Factor:	100	%
Annual Growth in Traffic Non - Timber :	1	%/year
Growth Factor for Non-Timber Traffic:	8.2857	
Annual Growth of Timber Traffic:	0	%/year
Growth Factor for Timber Traffic:	8	

FHWA Class	%ADT	ADT Per Class	% Annual Growth	Growth factor	T.F	18k ESAL*
1	0.3	10	1	8.2857	0.0004	6
2	66.1	2313	1	8.2857	0.0004	1,399
3	21.1	738	1	8.2857	0.0143	15,968
4	0.4	14	1	8.2857	0.1694	3,586
5	1.4	49	1	8.2857	0.1694	12,551
6	3.2	112	1	8.2857	0.3836	64,962
7	0.2	7	1	8.2857	0.3836	4,060
8	1	35	1	8.2857	0.8523	45,105
9a (Non-Timber)	5.3	160	1	8.2857	1.045	253,421
9b(Carrying Timber)		25	0	8.00000	3.385	248,233
10	0.7	24	1	8.2857	1.45	53,715
11	0.1	3	1	8.2857	1.84	9,738
12	0	0	1	8.2857	1.84	-
13	0.2	7	1	8.2857	1.84	19,475
	100	3500				732,221

***Design lane Traffic = ∑(Col.3)X(Col.5)X(Col.6)X(365)X(0.5)X(1.0)**

Design overlay for second performance period. Under Scenario 2, an overlay is designed for the second performance period using the AASHTO method of overlay design. According to the AASHTO method, the thickness of overlay was calculated as follow:

a. Flexible overlay on a flexible pavement

$$h_{ol} = \frac{SN_{ol}}{a_{ol}} = \frac{SN_y - F_{RL}SN_{xeff}}{a_{ol}} \qquad (3)$$

b. Flexible overlay over a rigid pavement, using visual condition factor method:

$$h_{ol} = \frac{SN_{ol}}{a_{ol}} = \frac{SN_y - F_{RL}(a_{2r}Do + SN_{xeff\ -\ rp})}{a_{ol}} \qquad (4)$$

Where,
H_{ol} = Overlay Thickness, inches
SN_{ol} = Required Structural Number of Overlay
SN_y = Total structural number required to support the overlay traffic over existing sub-grade conditions, calculated using the AASHTO flexible pavement design.
a_{ol} = Structural layer coefficient of HMA overlay
F_{RL} = Remaining life factor
SN_{xeff} = Total effective structural number of existing pavement structure above the sub-grade prior to overlay
a_{2r} = Structural Layer coefficient of existing cracked PCC pavement layer
D_o = Existing PCC layer thickness, inches
$SN_{xeff-rp}$ = Effective structural capacity of all of the remaining pavement layers above the sub-grade, except for the existing PCC layer

The value of SN_{xeff} was calculated with the pavement structural information prior to the design of overlay. For overlaying an existing pavement, the project staff assumed that two inches of the existing surface would be removed by milling immediately before the overlay was placed. The structural coefficient of the existing HMA materials was reduced to 0.33 to reflect the distressed condition of the pavement and its reduced structural capacity. A macro has been written to calculate the value of SN_y using the AASHTO design equation. Design lane ESALs were generated in the table 11 traffic data. The values for reliability and terminal serviceability were provided by the DOTD and vary with the functional classification of the road. Since US 84 is a rural major collector, reliability (R) is taken as 85 percent and the P_i and P_t values are taken as 4 and 2 respectively. The remaining life factor, F_{RL} was taken as 0.6. The overlay thickness was calculated as shown in the equation in table 12.

Table 12
Overlay design for US 84 under current condition (Scenario 2)
for second performance period

Existing Pavement

Layers	Thickness, in	Structural Coefficient	Drainage Factor	SN
1*	2.5	0.33	1	0.825
2	9	0.14	0.9	1.134
3	3.5	0.07	0.9	0.2205
			SN_{xeff}	2.1795

*** Thickness after milling 2"**

Overlay Material Design

Remaining Life Factor(F_{RL})	0.6
HMA a-value (a_{ol})	0.44
Roadbed Modulus, psi	9,176
Design Lane Traffic, ESALs	732,221
Reliability (%)	85
Overall Std. Deviation (So)	0.47
Initial PSI (p_i)	4
PSI at the end of Overlay (p_t)	2
Δ PSI	2
SNy	2.90
Overlay thickness	3.61
Wearing course thickness after overlay	6.11

$$h_{ol} = \frac{SN_{ol}}{a_{ol}} = \frac{SN_y - F_{RL} SN_{xeff}}{a_{ol}} = \frac{2.90 - 0.6(2.1795)}{0.44} = 3.61 \text{ Inches}$$

Traffic calculation for third performance period. Calculation of ESALs for the third performance period followed the same procedure as the second performance period. The

ADT for 2014 was calculated by multiplying the 2006 ADT times a growth factor for one percent growth per year for eight years. The distribution of non-timber traffic was assumed to remain constant during the eight-year performance period. Design lane ESALs were generated in table 13.

Table 13
Calculation of ESALs 2014 to 2022 under present GVW conditions (scenario2)

Timber on US 84, Rural Major Collector						
Performance Period:	8	years(Overlaid section)				
Average Daily Traffic in 2014	3790		Last Overlaid in	2014		
Directional Distribution Factor:	50	%				
Lane Distribution Factor:	100	%				
Annual Growth of Non - Timber Traffic:	1	%/year				
Growth Factor for Non-Timber Traffic:	8.2857					
Annual Growth of Timber Traffic:	0	%/year				
Growth Factor for Timber Traffic:	8					

FHWA Class	%ADT	ADT Per Class	% Annual Growth	Growth factor	T.F	18k ESAL*
1	0.3	11	1	8.2857	0.0004	7
2	66.1	2505	1	8.2857	0.0004	1,515
3	21.1	800	1	8.2857	0.0143	17,291
4	0.4	15	1	8.2857	0.1694	3,883
5	1.4	53	1	8.2857	0.1694	13,591
6	3.2	121	1	8.2857	0.3836	70,345
7	0.2	8	1	8.2857	0.3836	4,397
8	1	38	1	8.2857	0.8523	48,842
9a (Non-Timber)	5.3	176	1	8.2857	1.045	277,707
9b(Carrying Timber)		25	0	8.0000	3.385	248,233
10	0.7	27	1	8.2857	1.45	58,166
11	0.1	4	1	8.2857	1.84	10,544
12	0	0	1	8.2857	1.84	-
13	0.2	8	1	8.2857	1.84	21,089
	100	3790				775,611

***Design lane Traffic = ∑(Col.3)X(Col.5)X(Col.6)X(365)X(0.5)X(1.0)**

Overlay design for the third performance period. Determination of the overlay thickness for the third performance period followed the same procedure as described for the second performance period. The overlay thickness required for scenario 2 for the third performance period was 2.94 inches, as calculated in table 14.

Table 14
Overlay design for US 84 under current condition (Scenario 2) for third performance period

Existing Pavement

Layers	Thickness, in	Structural Coefficient	Drainage Factor	SN
1*	4.11	0.33	1	1.3557
2	9	0.14	0.9	1.134
3	3.5	0.07	0.9	0.2205
*** Thickness after milling 2"**			**SN_{xeff}**	2.710

Overlay Material Design

Remaining Life Factor(F_{RL})	0.6
HMA a-value (a_{ol})	0.44
Roadbed Modulus, psi	9,176
Design Lane Traffic, ESALs	775,611
Reliability (%)	85
Overall Std. Deviation (So)	0.47
Initial PSI (p_i)	4
PSI at the end of Overlay (p_t)	2
Δ PSI	2
SNy	
SN	2.92
Overlay thickness	2.94
Wearing course thickness after overlay	5.05

$$h_{ol} = \frac{SN_{ol}}{a_{ol}} = \frac{SN_y - F_{RL} SN_{xeff}}{a_{ol}} = \frac{2.92 - 0.6(2.721)}{0.44} = 2.94 \text{ Inches}$$

Calculation of net present worth for scenario 2. The overlays carried out on US 84 under the present conditions for the 20-year period between mid-1999 and mid-2019 are shown in figure 1. The net present worth (NPW) of these overlays was calculated for mid-1999 using an interest rate of 5 percent/year. The net present worth cost for the US 84 overlays, under present conditions (scenario 2) are

$$PW = OC_1\left(\frac{1}{(1+i_1)^{n1}}\right) + OC_2\left(\frac{1}{(1+i_1)^{n2}}\right) \tag{5}$$

$$PW = \$14{,}784\text{x}3.61\left(\frac{1}{(1+0.05)^{6.5}}\right) + \$14{,}784\text{x}2.94\left(\frac{1}{(1+0.05)^{14.5}}\right) = \$60{,}303$$

Where PW = Net Present Worth

OC_1 = Overlay Cost for the second performance period

$i_1 = i_2$ = The interest rate

OC_2 = Overlay Cost for third performance period

n_1 = number of years from the beginning of the study to the end of second performance period = 6.5 years

n_2 = number of years from the beginning of the study to the end of second performance period = 14.5 years

$14,784 represents the cost/ 12 ft lane mile.

OC_1 = $14,784 * overlay thickness in inches (based on 1999 statewide average cost of HMA)[2]

The overlay cost for the second performance period was ($14,784)*(3.61) = $53,343 and that for the third performance period was ($14,784)*(2.94) = $43,531.

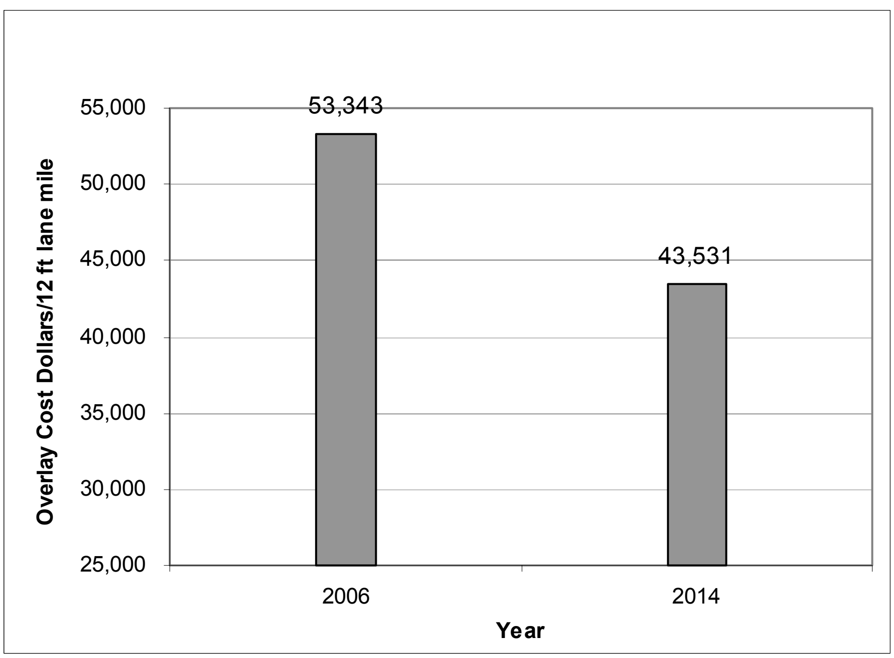

Figure 1
Overlay rehabilitation schedule for US 84 under present conditions (scenario 2)

Traffic calculation for remaining life of US 84 at the beginning of the analysis period (mid – 1999). To apply other GVW scenarios, the amount of traffic applied between 1998 when the last overlay was placed and the beginning of the analysis period must be calculated. The number of ESALs applied during this 1.5 year period is deducted from the design lane ESAL for which the overlay was designed. The value after the subtraction is the traffic to be served for the remaining life of the current overlay under each of the other GVW scenarios. The calculations of traffic applied between 1998 and mid-1999 is shown in table 15.

Table 15
Calculation of design traffic on US 84 that was applied under scenario 2 between 1998 and mid-1999

Timber on US 84, Rural Major Collector						
Year of last overlay		1998	Year when Study was conducted:		1999.5	
ADT/AADT:		3281	Period Since Last Overlay:		1.5	
Directional Distribution Factor:		50	%			
Lane Distribution Factor:		100	%			
Annual Growth in Non Timber Traffic:		1	%/Year			
Growth Factor for Non-Timber Traffic:		1.5037				
Annual Growth in Timber Traffic:		0	%/Year			
Growth Factor for Timber Traffic:		1.5				
FHWA Class	%ADT	ADT Per Class	% Annual Growth	Growth factor	T.F	18k ESAL
1	0.3	10	1	1.5037	0.0004	1
2	66.1	2168	1	1.5037	0.0004	238
3	21.1	692	1	1.5037	0.0143	2,716
4	0.4	13	1	1.5037	0.1694	610
5	1.4	46	1	1.5037	0.1694	2,135
6	3.2	105	1	1.5037	0.3836	11,051
7	0.2	7	1	1.5037	0.3836	691
8	1	33	1	1.5037	0.8523	7,673
9a (Non-Timber)	5.3	149	1	1.5037	1.045	42,661
9b(Carrying Timber)		25	0	1.5000	3.385	46,544
10	0.7	23	1	1.5037	1.45	9,138
11	0.1	3	1	1.5037	1.84	1,657
12	0	0	1	1.5037	1.84	-
13	0.2	7	1	1.5037	1.84	3,313
	100	3281				128,429

***Design lane Traffic = $\sum$(Col.3)X(Col.5)X(Col.6)X(365)X(0.5)X(1.0)**

Calculation of number of years required by scenario 1 to use the remaining design traffic in first performance period. For a timber truck at 80,000 lb GVW, the following axle configuration and ESALs are obtained from tables D1 and D2 of AASHTO [3] with SN=4 and $P_t = 2.0$;

Steering Axle (12,000 lbs.) = 0.183
Tandem Axle (34,000 lbs.) = 1.08
Tandem Axle (34,000 lbs.) = 1.08
ESALs for each Truck = 2.343 ESALs
Maximum Payload per truck = GVW – tare weight of truck
= 80,000 – 26,600 = 53,400 lbs.
Hence, the number of trucks required to carry the timber under scenario 2

$$= \frac{550{,}000{,}000}{53{,}400} = 10{,}300 \text{ trucks/year} = 28 \text{ trucks/day}$$

A simulation was run in excel to find the number of years required for the scenario 1 traffic to equal the remaining ESALs in the 1998 overlay designed for scenario 2. The results presented in table 16 show that under scenario 1, where timber is carried by 28 trucks/day, a little over 7 years is required to use the remaining design ESALs. Notice in table 16, that in 7.09 years the scenario 1 traffic produces 563,991 ESALs, which is slightly larger than the 563,722 ESALs remaining life for the scenario 2 overlay.

Table 16
Calculation of number of years required by Scenario one to use the remaining design traffic in first performance period

Timber on US 84, Rural Major Collector

Performance Period:	7.09	Years	Scenario 2 Remaining Life ESALs:	563,722
ADT/AADT:	3281		year:	1999.5
Directional Distribution Factor:	50	%		
Lane Distribution Factor:	100	%		
Annual Growth in Traffic:	1	%/year		
Growth Factor for Non-Timber Traffic:	7.31			
Annual Growth in Timber Traffic:	0	%/year		
Growth Factor for Timber Traffic:	7.09			

FHWA Class	%ADT	ADT Per Class	% Annual Growth	T.F	Growth Factor	18k ESAL
1	0.3	10	1	0.0004	7.31	5
2	66.1	2168	1	0.0004	7.31	1,155
3	21.1	692	1	0.0143	7.31	13,185
4	0.4	13	1	0.1694	7.31	2,961
5	1.4	46	1	0.1694	7.31	10,364
6	3.2	105	1	0.3836	7.31	53,642
7	0.2	7	1	0.3836	7.31	3,353
8	1	33	1	0.8523	7.31	37,245
9a Non Timber Trucks	5.3	146	1	1.045	7.31	202,749
10	0.7	23	1	1.45	7.31	44,355
11	0.1	3	1	1.84	7.31	8,041
12	0	0	1	1.84	7.31	-
13	0.2	7	1	1.84	7.31	16,081
9b Timber Trucks		28	0	2.343	7.09	170,855
	100	3281				563,991

Years Simulator

Number of Years required to reach Scenario 2 ESALs 7.09

Calculation of ESAL for scenario 1 second performance period. Since scenario 1 traffic uses the remaining design ESALs by mid-2006, the overlay for the second performance period carries traffic generated from 2006 to 2014. ESAL calculations similar to those generated for scenario 2 were generated for scenario 1 and are included in table 17. The traffic was projected using the traffic growth factors. The ADT for the beginning of the second performance period was generated by multiplying 1 percent growth per year for 8 years to get 3,520 vehicles per day.

Table 17
Calculation of ESALs starting in mid-2006 to 2014 under scenario 1

Timber on US 84, Rural Major Collector

Performance Period:	8.00	years(Overlaid Section)	
ADT/AADT:	3520	Last Overlaid in	**2006.6**
Directional Distribution Factor:	50	%	
Lane Distribution Factor:	100	%	
Annual Growth of Non - Timber Traffic:	1.00	%/year	
Growth Factor for Non-Timber Traffic:	8.2857		
Annual Growth for Timber Traffic:	0.00	%/year	
Growth Factor for Timber Traffic:	8.00		

FHWA Class	%ADT	ADT Per Class	% Annual Growth	Growth factor	T.F	18k ESAL
1	0.3	11	1	8.2857	0.0004	6
2	66.1	2327	1	8.2857	0.0004	1,407
3	21.1	743	1	8.2857	0.0143	16,062
4	0.4	14	1	8.2857	0.1694	3,607
5	1.4	49	1	8.2857	0.1694	12,625
6	3.2	113	1	8.2857	0.3836	65,345
7	0.2	7	1	8.2857	0.3836	4,084
8	1	35	1	8.2857	0.8523	45,371
9a (Non-Timber)	5.3	159	1	8.2857	1.045	250,587
9b(Carrying Timber)		28	0	8.0000	2.343	191,564
10	0.7	25	1	8.2857	1.45	54,032
11	0.1	4	1	8.2857	1.84	9,795
12	0	0	1	8.2857	1.84	-
13	0.2	7	1	8.2857	1.84	19,590
	100	3520				674,074

***Design lane Traffic = ∑(Col.3)X(Col.5)X(Col.6)X(365)X(0.5)X(1.0)**

Overlay design for the second performance period. Determination of the overlay thickness for the second performance period in scenario 1 followed the same procedure as described earlier in scenario 2. The overlay thickness required for scenario 1 for the second performance period was 3.52 inches, as calculated in table 18.

Table 18
Overlay design for US 84 under scenario 1 for the second performance period

Existing Pavement

Layers	Thickness, in	Structural Coefficient	Drainage Factor	SN
1*	2.5	0.33	1	0.825
2	9	0.14	0.9	1.134
3	3.5	0.07	0.9	0.2205
*** Thickness after milling 2"**			**SN_{xeff}**	2.1795

Overlay Material Design

Remaining Life Factor(F_{RL})	0.6
HMA a-value (a_{ol})	0.44
Roadbed Modulus, psi	9,176
Design Lane Traffic, ESALs	674,074
Reliability (%)	85
Overall Std. Deviation (So)	0.47
Initial PSI (p_i)	4
PSI at the end of Overlay (p_t)	2
Δ PSI	2
SNy	
SN	2.86
Overlay thickness	3.52
Wearing course thickness after overlay	6.02

$$h_{ol} = \frac{SN_{ol}}{a_{ol}} = \frac{SN_y - F_{RL} SN_{xeff}}{a_{ol}} = \frac{2.86 - 0.6(2.1795)}{0.44} = 3.52 \text{ Inches}$$

Traffic calculation (mid-2014 to mid-2022) for the third performance period. Calculation of ESALs for the third performance period follows the same procedure as for the second performance period. The ADT for 2014 was calculated by multiplying the 2006 ADT by a growth factor of one percent growth per year for eight years. The distribution of non-timber traffic is assumed to remain constant during the eight-year performance period. The design lane ESALs are generated in table 19.

Table 19
Calculation of ESALs starting in mid-2014 to 2022 under scenario 1

Timber on US 84, Rural Major Collector

Performance Period:	8	years(Overlaid Section)	
ADT/AADT:	3812	Last Overlaid in	2014.6
Directional Distribution Factor:	50	%	
Lane Distribution Factor:	100	%	
Annual Growth of Non - Timber Traffic:	1	%/year	
Growth Factor for Non-Timber Traffic:	8.2857		
Annual Growth for Timber Traffic:	0	%/year	
Growth Factor for Timber Traffic:	8		

FHWA Class	%ADT	ADT Per Class	% Annual Growth	Growth factor	T.F	18k ESAL
1	0.3	11	8	8.2857	0.0004	7
2	66.1	2520	8	8.2857	0.0004	1,524
3	21.1	804	8	8.2857	0.0143	17,393
4	0.4	15	8	8.2857	0.1694	3,906
5	1.4	53	8	8.2857	0.1694	13,671
6	3.2	122	8	8.2857	0.3836	70,759
7	0.2	8	8	8.2857	0.3836	4,422
8	1	38	8	8.2857	0.8523	49,130
9a (Non-Timber)	5.3	174	8	8.2857	1.045	275,016
9b(Carrying Timber)		28	0	8.0000	2.343	191,564
10	0.7	27	8	8.2857	1.45	58,509
11	0.1	4	8	8.2857	1.84	10,606
12	0	0	8	8.2857	1.84	-
13	0.2	8	8	8.2857	1.84	21,213
	100	3812				717,720

***Design lane Traffic = ∑(Col.3)X(Col.5)X(Col.6)X(365)X(0.5)X(1.0)**

Overlay design for the third performance period. Calculation of the overlay thickness for the third performance period is presented in table 20. The overlay thickness required for scenario 1 for the third performance period was 2.90 inches.

Table 20
Overlay design for US 84 under scenario 1 for third performance period

Existing Pavement

Layers	Thickness, in	Structural Coefficient	Drainage Factor	SN
1*	4.02	0.33	1	1.328
2	9	0.14	0.9	1.134
3	3.5	0.07	0.9	0.2205
*** Thickness after milling 2"**			**SN_{xeff}**	2.682

Overlay Material Design

Remaining Life Factor(F_{RL})	0.6
HMA a-value (a_{ol})	0.44
Roadbed Modulus, psi	9,176
Design Lane Traffic, ESALs	717,720
Reliability (%)	85
Overall Std. Deviation (So)	0.47
Initial PSI (p_i)	4
PSI at the end of Overlay (p_t)	2
Δ PSI	2
SNy	2.89
Overlay thickness	2.90
Wearing course thickness after overlay	4.93

$$h_{ol} = \frac{SN_{ol}}{a_{ol}} = \frac{SN_y - F_{RL} SN_{xeff}}{a_{ol}} = \frac{2.89 - 0.6(2.695)}{0.44} = 2.90 \text{ Inches}$$

Calculation of net present worth for scenario 1. The overlays carried out on US 84 under the present conditions for the 20-year period from mid-1999 to mid-2019 are shown in figure 2. The net present worth (NPW) of these overlays was calculated for mid-1999 using an interest rate of five percent/year. The net present worth cost for the US 84 overlays, under scenario 1 is $36,852 for the second performance period and $20,548 for the third performance period for a total NPW cost of $57,400 per 12 ft. lane mile.

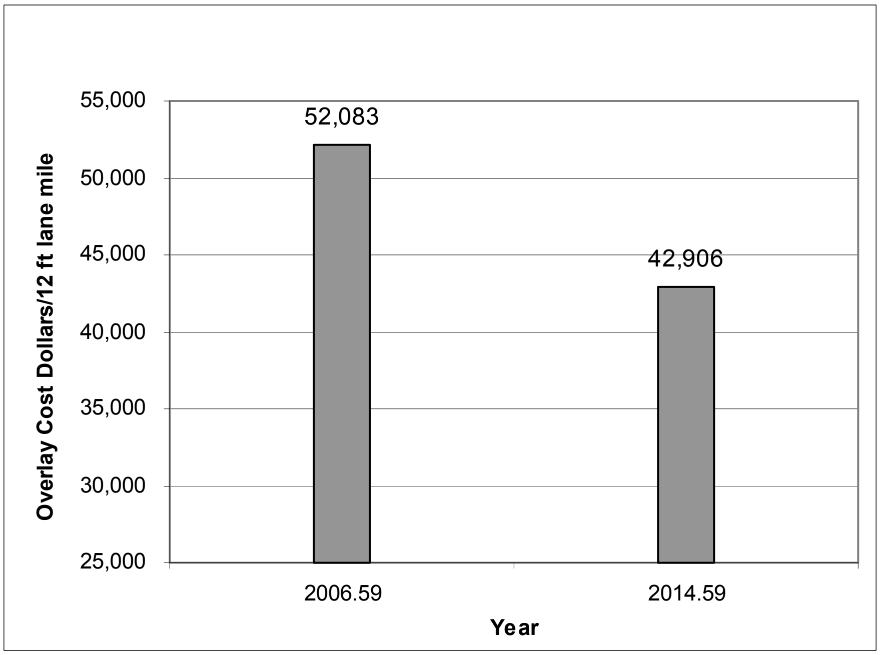

Figure 2
Overlay rehabilitation schedule for US 84 under scenario one

OC_1 = 3.52 * 14,784 = $ 52,083 /12 ft. lane mile
OC_2 = 2.90 * 14,784 = $ 42,906 /12ft. lane mile

Calculation of number of years required for scenario 3 traffic to use the remaining design traffic ESALs in the first performance period. In scenario 3, a GVW of 100,000 lbs. is used for timber transport on the FHWA class 9 trucks. For a timber truck at 100,000 lb. GVW, the following axle configuration and ESALs are obtained from tables D1 and D2 of AASHTO [3] with SN=4 and P_t = 2.0;

Steering Axle (12,000 lbs) = 0.183
Tandem Axle (44,000 lbs) = 3.18
Tandem Axle (44,000 lbs) = 3.18
ESALs for each Truck = 6.543 ESALs

Maximum Payload per truck = GVW – tare weight of truck

= 100,000 – 26,600 = 73,400lbs.

Hence Number of trucks required to carry the timber under scenario 3

$$= \frac{550,000,000}{73,400} = 7493 \text{ trucks/year} = 21 \text{ trucks/day}$$

A simulation was run in Excel to find the number of years required for the scenario 3 traffic to equal the remaining ESALs in the 1998 overlay design traffic under scenario 2. The results presented in table 21 shows that under scenario 3, in which timber is carried by 21 trucks/day, almost 5.35 years are required to use the remaining design ESALs. As seen in scenario 1, scenario 3 produces 563,747 ESALs in 5.35 years, which is slightly larger than the 563,722 ESALs for the scenario 2 overlay design.

Table 21
Calculation of number of years required by scenario three to use the remaining design traffic in first performance period

Timber on US 84, Rural Major Collector

Performance Period:	5.35	Years	Scenario 2 Remaining Life ESALs:	563,722
ADT/AADT:	3281		year:	1999.5
Directional Distribution Factor:	50	%		
Lane Distribution Factor:	100	%		
Annual Growth in Traffic:	1	%/year		
Growth Factor for Non-Timber Traffic:	5.47			
Annual Growth in timber Traffic:	0	%/year		
Growth Factor for Timber Traffic:	5.35			

FHWA Class	%ADT	ADT Per Class	% Annual Growth	T.F	% Growth in T.F	18k ESAL
1	0.3	10	1	0.0004	5.47	4
2	66.1	2168	1	0.0004	5.47	864
3	21.1	692	1	0.0143	5.47	9,858
4	0.4	13	1	0.1694	5.47	2,214
5	1.4	46	1	0.1694	5.47	7,749
6	3.2	105	1	0.3836	5.47	40,106
7	0.2	7	1	0.3836	5.47	2,507
8	1	33	1	0.8523	5.47	27,847
9a Non Timber Traffic	5.3	153	1	1.045	5.47	159,592
10	0.7	23	1	1.45	5.47	33,163
11	0.1	3	1	1.84	5.47	6,012
12	0	0	1	1.84	5.47	-
13	0.2	7	1	1.84	5.47	12,024
9b Timber Traffic		21	0	6.543	5.35	261,809
	100	3281				563,747

Year simulator

Number of Years required to reach Scenario 2 ESALs 5.35

Calculation of ESAL for scenario 3 from 2004.9 to 2012.9 for second performance period. Since the scenario 3 traffic uses the remaining design ESALs at the end of 2004, the second performance period carries traffic generated between 2004.9 to 2012.9. ESAL calculations similar to those generated for the scenario 2 are generated for scenario 3 and included in table 22. The traffic was projected using the traffic growth factors. The ADT for the beginning of the second performance period is generated by multiplying 1 percent growth per year for 8 years or 3,460 vehicles per day.

Table 22
Calculation of ESALs starting in 2004.9 to 2012.9 under scenario 3

Timber on US 84, Rural Major Collector

Performance Period:	8.00	years(Overlaid Section)
ADT/AADT:	3460	Last Overlaid in **2004.9**
Directional Distribution Factor:	50	
Lane Distribution Factor:	100	
Annual Growth of Non - Timber Traffic:	1.00	
Growth Factor for Non-Timber Traffic:	8.2857	
Annual Growth for Timber Traffic:	0.00	
Growth Factor for Timber Traffic:	8	

FHWA Class	%ADT	ADT Per Class	% Annual Growth	Growth factor	T.F	18k ESAL
1	0.3	10	8	8.2857	0.0004	6
2	66.1	2287	8	8.2857	0.0004	1,383
3	21.1	730	8	8.2857	0.0143	15,786
4	0.4	14	8	8.2857	0.1694	3,545
5	1.4	48	8	8.2857	0.1694	12,408
6	3.2	111	8	8.2857	0.3836	64,223
7	0.2	7	8	8.2857	0.3836	4,014
8	1	35	8	8.2857	0.8523	44,592
9a (Non-Timber)	5.3	162	8	8.2857	1.045	256,588
9b(Carrying Timber)		21	0	8.0000	6.543	401,217
10	0.7	24	8	8.2857	1.45	53,104
11	0.1	3	8	8.2857	1.84	9,627
12	0	0	8	8.2857	1.84	-
13	0.2	7	8	8.2857	1.84	19,254
	100	3460				885,747

***Design lane Traffic = ∑(Col.3)X(Col.5)X(Col.6)X(365)X(0.5)X(1.0)**

Overlay design for the second performance period. Calculation of the overlay thickness for the second performance in scenario 3 is presented in table 23. The overlay thickness required for scenario 3 was 3.81 inches.

Table 23
Overlay design for US 84 under scenario 3 for second performance period

Existing Pavement

Layers	Thickness, in	Structural Coefficient	Drainage Factor	SN
1*	2.5	0.33	1	0.825
2	9	0.14	0.9	1.134
3	3.5	0.07	0.9	0.2205
*** Thickness after milling 2"**			**SN_{xeff}**	2.1795

Overlay Material Design

Remaining Life Factor(F_{RL})	0.6
HMA a-value (a_{ol})	0.44
Roadbed Modulus, psi	9,176
Design Lane Traffic, ESALs	885,747
Reliability (%)	85
Overall Std. Deviation (So)	0.47
Initial PSI (p_i)	4
PSI at the end of Overlay (p_t)	2
Δ PSI	2

SNy

SN	2.98
Overlay thickness	3.81
Wearing course thickness after overlay	6.31

$$h_{ol} = \frac{SN_{ol}}{a_{ol}} = \frac{SN_y - F_{RL} SN_{xeff}}{a_{ol}} = \frac{2.98 - 0.6(2.1795)}{0.44} = 3.81 \quad \text{Inches}$$

Traffic calculation for the third performance period. Calculation of ESALs for the third performance period followed the same procedure as the second performance period. The ADT for 2014 was calculated by multiplying the 2006 ADT by a growth factor for one percent growth per year for eight years. The distribution of non-timber traffic is assumed to remain constant during the eight-year performance period. The design lane ESALs are generated in table 24.

Table 24
Calculation of ESALs starting in 2012.9 to 2020.9 under scenario 3

Timber on US 84, Rural Major Collector

Performance Period:	8.00	years(Overlaid Section)	
ADT/AADT:	3747	Last Overlaid in	**2012.9**
Directional Distribution Factor:	50		
Lane Distribution Factor:	100		
Annual Growth of Non - Timber Traffic:	1.00		
Growth Factor for Non-Timber Traffic:	8.29		
Annual Growth for Timber Traffic:	0.00		
Growth Factor for Timber Traffic:	8		

FHWA Class	%ADT	ADT Per Class	% Annual Growth	Growth factor	T.F	18k ESAL
1	0.3	11	8	8.2857	0.0004	7
2	66.1	2477	8	8.2857	0.0004	1,498
3	21.1	791	8	8.2857	0.0143	17,094
4	0.4	15	8	8.2857	0.1694	3,839
5	1.4	52	8	8.2857	0.1694	13,436
6	3.2	120	8	8.2857	0.3836	69,545
7	0.2	7	8	8.2857	0.3836	4,347
8	1	37	8	8.2857	0.8523	48,287
9a (Non-Timber)	5.3	199	8	8.2857	1.045	313,781
9b(Carrying Timber)		21	0	8.0000	6.543	401,217
10	0.7	26	8	8.2857	1.45	57,504
11	0.1	4	8	8.2857	1.84	10,424
12	0	0	8	8.2857	1.84	-
13	0.2	7	8	8.2857	1.84	20,849
	100	3768				961,827

***Design lane Traffic = ∑(Col.3)X(Col.5)X(Col.6)X(365)X(0.5)X(1.0)**

Overlay design for the third performance period. Determination of the overlay thickness for the third performance period followed the same procedure as described for the second performance period. The overlay thickness required for scenario 1 for the third performance period was 3.08 inches as calculated in table 25.

Table 25
Overlay design for US 84 under scenario 3 for third performance period

Existing Pavement

Layers	Thickness, in	Structural Coefficient	Drainage Factor	SN
1*	4.31	0.33	1	1.4217
2	9	0.14	0.9	1.134
3	3.5	0.07	0.9	0.2205
*** Thickness after milling 2"**			**SN_{xeff}**	2.776

Overlay Material Design

Remaining Life Factor(F_{RL})	0.6
HMA a-value (a_{ol})	0.44
Roadbed Modulus, psi	9,176
Design Lane Traffic, ESALs	961,827
Reliability (%)	85
Overall Std. Deviation (So)	0.47
Initial PSI (p_i)	4
PSI at the end of Overlay (p_t)	2
Δ PSI	2
SN	3.02
Overlay thickness	3.08
Wearing course thickness after overlay	5.39

SNy

$$h_{ol} = \frac{SN_{ol}}{a_{ol}} = \frac{SN_y - F_{RL} SN_{xeff}}{a_{ol}} = \frac{3.02 - 0.6(2.788)}{0.44} = 3.08 \text{ Inches}$$

Calculation of net present worth for scenario 3. The overlays carried out on US 84 under the present conditions for the 20-year period between mid-1999 and mid-2019 are shown in figure 3. The net present worth (NPW) of these overlays was calculated for mid-1999 using an interest rate of five percent /year. The net present worth cost for the US 84 overlays under scenario 3 is $43,366 for the second performance period and $23,760 for the third performance period for a total cost of $67,126 per 12 ft. lane mile.

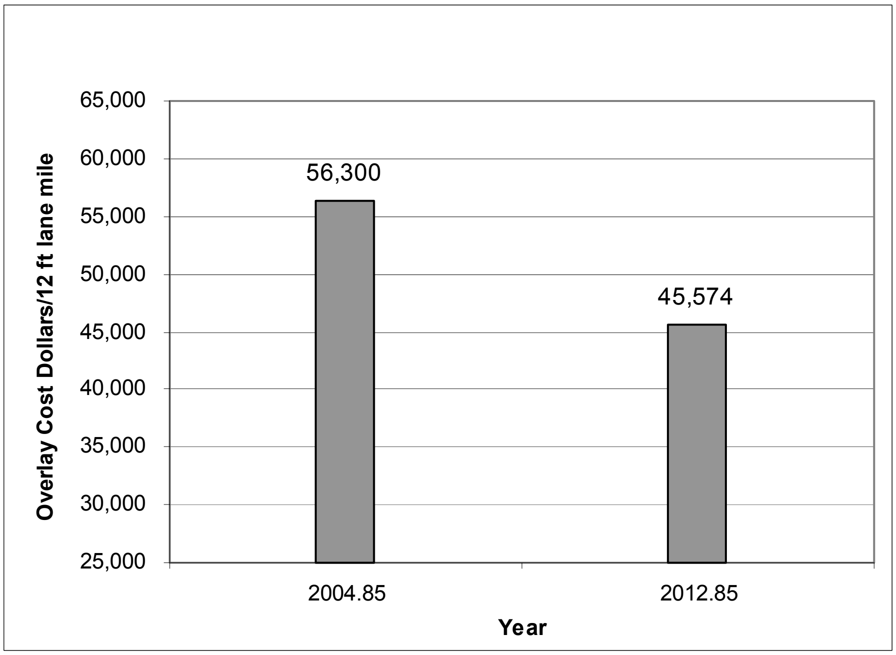

Figure 3
Overlay rehabilitation schedule for US84 under scenario 3

OC_1 = 3.81 * 14,784 = $ 56,300 per 12 ft. lane mile
OC_2 = 3.08 * 14,784 = $ 45,574 per 12 ft. lane mile

Comparison of net present worth among the 3 Scenarios. Table 26 contains the net present worth of the DOTD cost to rehabilitate one 12-ft. lane mile of US 84 for each of the three scenarios. Notice that as the GVW increases, the overlay thicknesses and their subsequent costs increase. One of the critical issues from the DOTD viewpoint is whether or not the fees paid by the timber haulers pay for the increased cost incurred by the DOTD. To provide part of the answer to this question, the added cost to carry these heavier loads must be considered as well as the value of the permit paid by trucks transporting timber. The extra cost incurred by the DOTD for the 12-ft. lane mile between scenario 2 and scenario 1 is $2,903, or $5,806 for one centerline mile of US 84.

Table 26
Comparison of overlay thickness, cost and net present worth of the GVW scenarios

Scenario	GVW, lbs	Thickness, in		Cost/ 12 ft lane mile		Net present worth at 5%/year
		Overlay 1	Overlay 2	Overlay 1	Overlay 2	
1	80,000	3.52	2.90	52,083	42,906	57,400
2	86,600	3.61	2.94	53,343	43,531	60,303
3	100,000	3.81	3.08	56,300	45,574	67,126

Evaluation of Bridge Costs

The methodology used in the analysis phase evaluated the effect of the heavy loads on the bridges from the trucks transporting forestry products, Louisiana-produced lignite coal, and coke fuel, based on LRFD and LFD design recommendations. The demand on the bridge girders due to the heavy truck loads was calculated based on inventory information on span type and geometry, i.e., simple span, continuous span, total length, length of main span, and number of approach spans. Finite element analysis was used in this task of the research.

The effects of hauling timber, lignite coal, and coke fuel on Louisiana bridges were determined by comparing the flexural, shear, and serviceability conditions of the bridges under their design load to the conditions under the 3S2 Truck configuration as shown in figure 4. The bridge analysis methodology was discussed in the PRC meetings on July 29 and September 2, 2004. A simplified method based on AASHTO design guidelines was determined to be the most prudent approach to meet the short and strict schedule for this study.

The short and long term effects of the timber and lignite coal truck loads were determined based on the ratio of the maximum moments, shear forces, or deflection for each bridge in the sample. The AASHTO Line Girder Analysis approach, detailed analysis using finite element models, and GTSTRUDL Software were used. The design load for the bridge, as listed in the bridge inventory, was used. The truck loads for hauling timber and lignite coal were based on the 3S2 truck configuration, with maximum tandem load of 48,000 lb. and steering axle of 12,000 lb.

The first step in the analysis used the influence line procedures to determine the critical location of the trucks on the bridges that would result in maximum moment and shear forces. Based on the results from the influence line analyses, the effects of the loads on the bridge girders and bridge decks were determined. Also, the magnitude of the maximum moment and shear forces were calculated. Next, the ratios of the results for the 3S2 truck and the design truck (H15 or HS20-44) for flexural and shear forces or stresses were calculated. The serviceability criteria were evaluated for simply supported girders based on their deflections.

The selected bridges (State 1881, and Parish 945) listed in Appendix A table 1 were grouped into six different categories based on their design approach. These categories were:

- Simple beam
- Continuous beam
- Culvert
- Others
- Posted bridges
- Design load low (5, 10 ton)

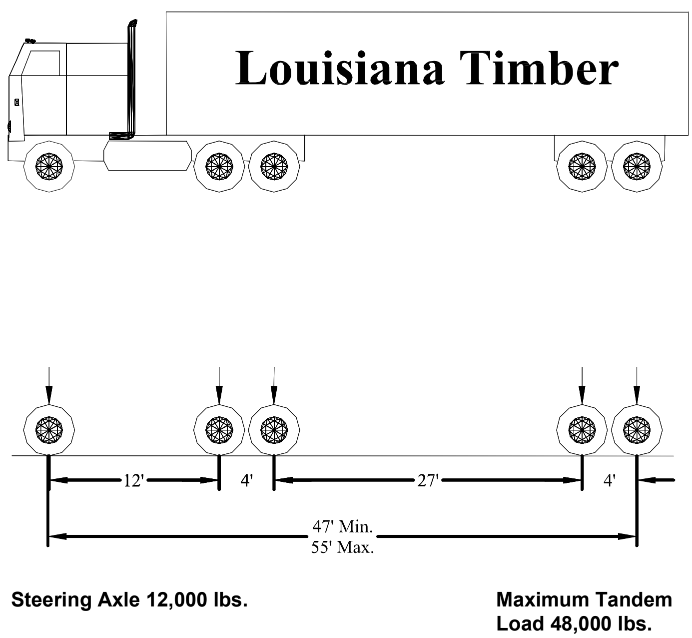

Figure 4
Truck 3S2 hauling timber or lignite coal on Louisiana bridges

During the PRC meeting on September 2, 2004, these bridge categories were discussed and the PRC recommended and approved that the analyses in this study would focus only on the two categories (simple beam and continuous beam).

The analysis for bridges in the "simple beam" category was performed using spread sheets to calculate the maximum moment along with shear and deflection for all the spans in the sample for this study. The ratios for the flexural, shear, and deflection due to the design load and the 3S2 truck load were calculated. All calculations pertaining to this category are included in the appendices of this report.

The analysis for bridges in the "continuous beam" category was performed using GTSTRUDL to develop the influence lines for moment (positive and negative) and shear forces. These results were used in spread sheets to determine the critical location for the design truckload and the 3S2 truck. Then, the maximum moments and shear forces were calculated.

Identify the Critical Bridges for the Study

The critical bridges for this study were considered to be those located on the roads most traveled by the trucks hauling timber or lignite. The roads considered were Louisiana State Highways, U.S. Numbered Roads, and Interstate Highways. The review and selection processes were based on two factors: (1) the amount of timber harvest each parish produces; and (2) the parish's geographic location.

Trucks hauling timber (State bridges). The 11 parishes that were selected reported more than $30 million in income from their 2003 timber harvest, as shown in the figure 5. These parishes are located north of Interstate Highway I-10.

The control section numbers for roads heavily traveled by timber and lignite trucks were identified in this study. The roads that are located in the parishes shown in figure 5 were used in the bridge inventory database to identify the critical bridges for this study. As shown in table 1 in Appendix A, 1,872 state bridges are located on the roads most traveled by trucks hauling timber. These bridges are used in the analysis phase of this study.

Trucks hauling lignite coal. The lignite coal is transported between Oxbow and Dolet Hills Power Plant. The trucks use LA 1, US 84, and LA 3248. The bridges on this route were identified from the bridge inventory database. Nine bridges are included in the analysis phase of this study.

Trucks hauling timber (Parish bridges). The critical parish bridges were identified as bridges located on parish roads that connect to the LA State Highways, U.S. Numbered Roads, and Interstate Highways most traveled by trucks hauling timber. The DOTD personnel identified 945 parish bridges that were included in Appendix A, Table 1.

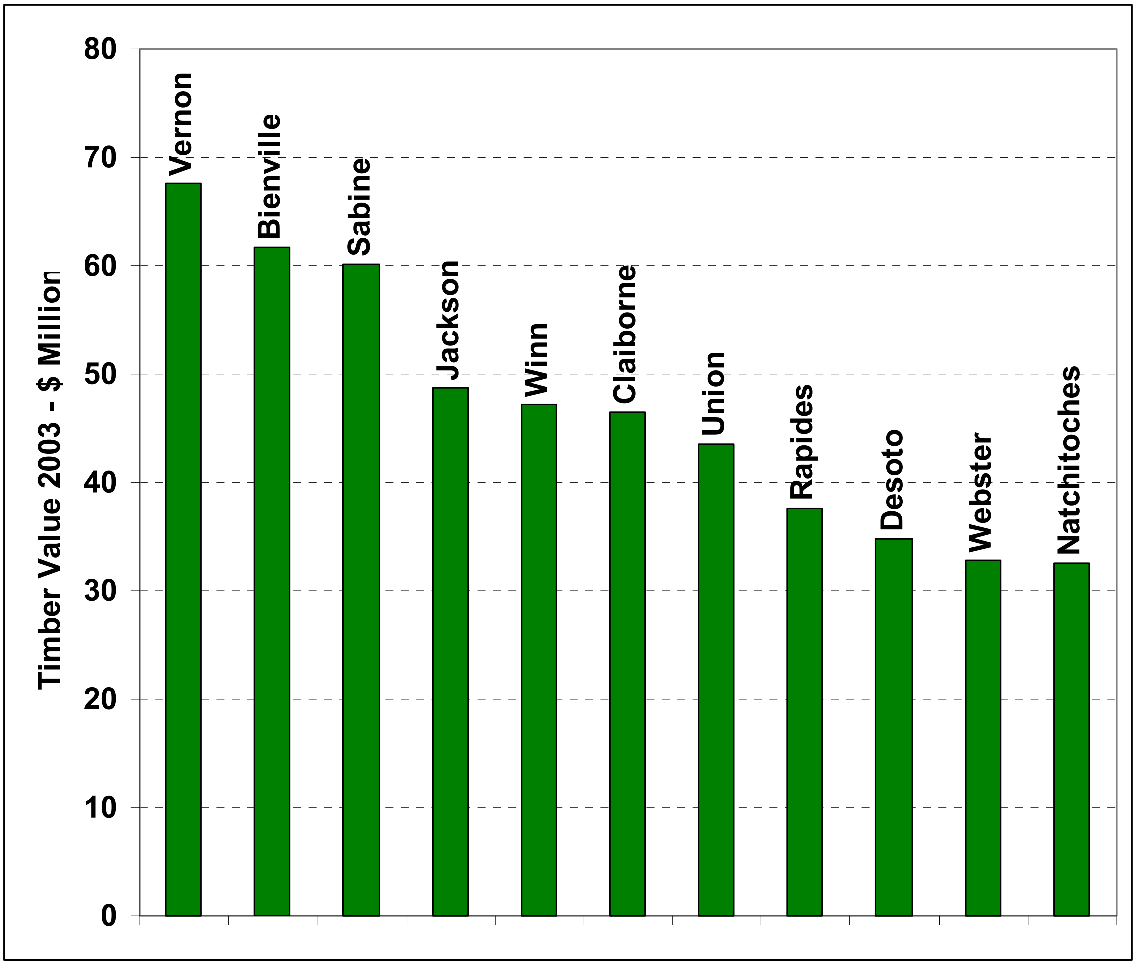

Figure 5
Location of bridges included in this study

Analysis for Bridge Girders

Influence line analysis

When the truck loads, performed as the concentrated loads, were placed on the bridge deck, an influence surface could be generated. Instead of using the influence surfaces to find the critical moments, shear, and deflection under certain load conditions, the influence line was used. The bending moment and shear for which the influence line was to be determined was computed as a unit load placed at different positions over the length and the width of the bridge. The maximum deflection was computed by superposition.

In this study, the H15 truck loads, HS20-44 truck loads, and 3S2 truck loads were used in the analysis procedure. Both hand calculations and computer models in GTSTRUDL were used to determine the critical load location and the corresponding moment and shear forces. Also, associated deflections and stresses in the bridge girders and bridge decks were determined.

Simple Span Bridges

The influence line analysis for bridges with simple spans was performed using hand calculations and spread sheets. The standard truck configurations for H15 and HS20-44, as provided in AASHTO Chapter 3, were used. The trucks that haul timber and lignite coal in Louisiana were similar to the Type 3S2 truck configuration shown in figure 4. The span length for bridge girders between 20 ft. and 120 ft. (at 1 ft. increments) were considered for this study. All truck loads were placed on each girder as shown in figures 6, 7 and 8.

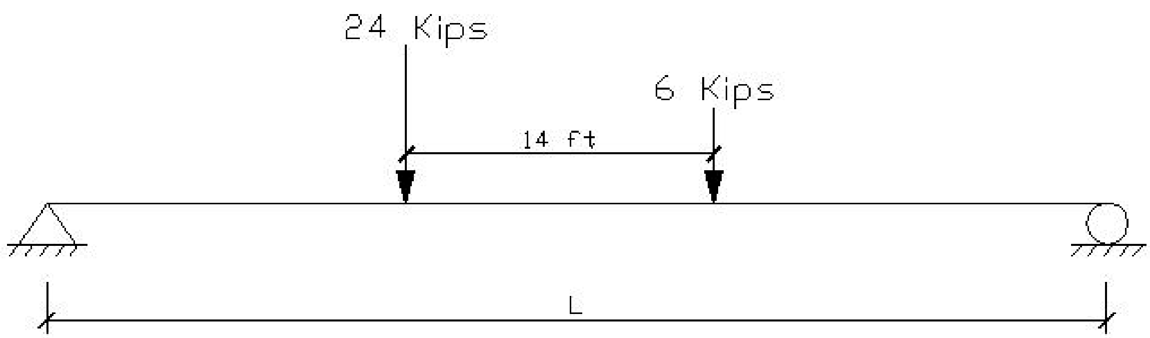

Figure 6
H15 truck loads on simple span bridge girders

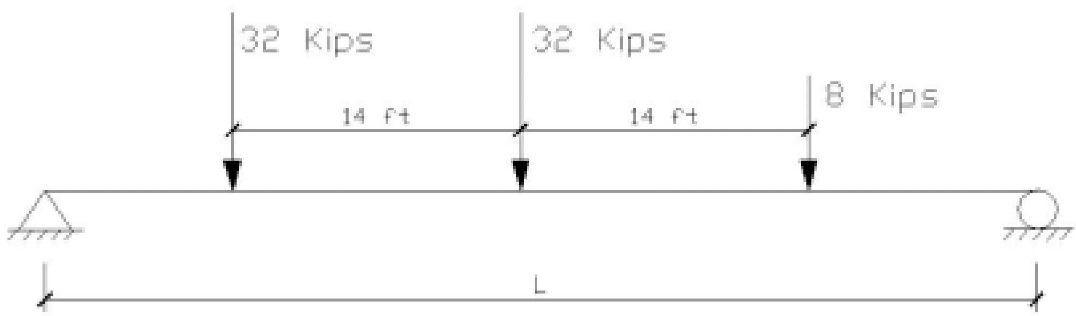

Figure 7
HS20-44 truck loads on simple span bridge girders

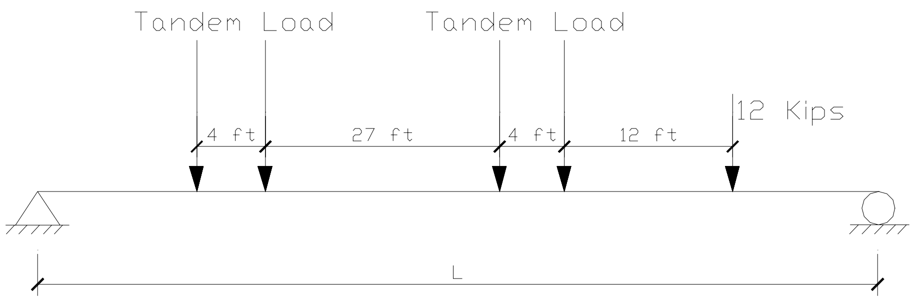

Figure 8
3S2 truck loads on simple span bridge girders

The loads were moved on the bridge girder at 1 ft. increments to calculate the absolute maximum moment and shear forces. The different load conditions for the corresponding girder span lengths are shown in Appendix B, table 1.

Absolute maximum shear, moment, and deflection. The absolute maximum shear in simply supported bridge girders occurred next to the supports. Therefore, the loads were positioned so that the first wheel load in sequence was placed close to the support.

The absolute maximum moment in simply supported bridge girders occurred under one of the concentrated forces. This force was positioned on the beam so that it and the resultant force of the system were equidistant from the girder's centerline.

The truck location on the bridge girder that caused the maximum absolute moment was used to determine the maximum deflection.

Also, the uniform lane load of 0.48kip/ft. as provided in AASHTO LRFD Specifications for Highway Bridges was considered. Lane load controlled some of the design conditions for the H15 truck loads. Tables 2, 3 and 4 in Appendix B summarize the results for the absolute maximum moment, shear, and deflection, for the H15, HS20-44, and 3S2 truck configurations. Cases where lane loads controlled the design were identified.

Continuous Span Bridges

The influence line analysis was performed using GTSTRUDL software. The bridge girder models were considered as three equal spans. The first support for the girder was considered pin support and the remaining three supports were roller type. The span lengths considered for this study varied from 20 ft. to 130 ft. (at 5 ft. increments). All truck loads were placed on each girder, as previously shown in figures 6, 7 and 8.

Modeling in GTSTRUDL. GTSTRUDL software was used to calculate the influence line of moment and shear at each joint along the length of the bridge girder. Due to the symmetry of the bridge, only the left half part of the bridge girder was considered. The truck loads were applied in both directions, from left to right and from right to left, as shown in figure 9. The results were used in the following steps to calculate the moment and shear forces.

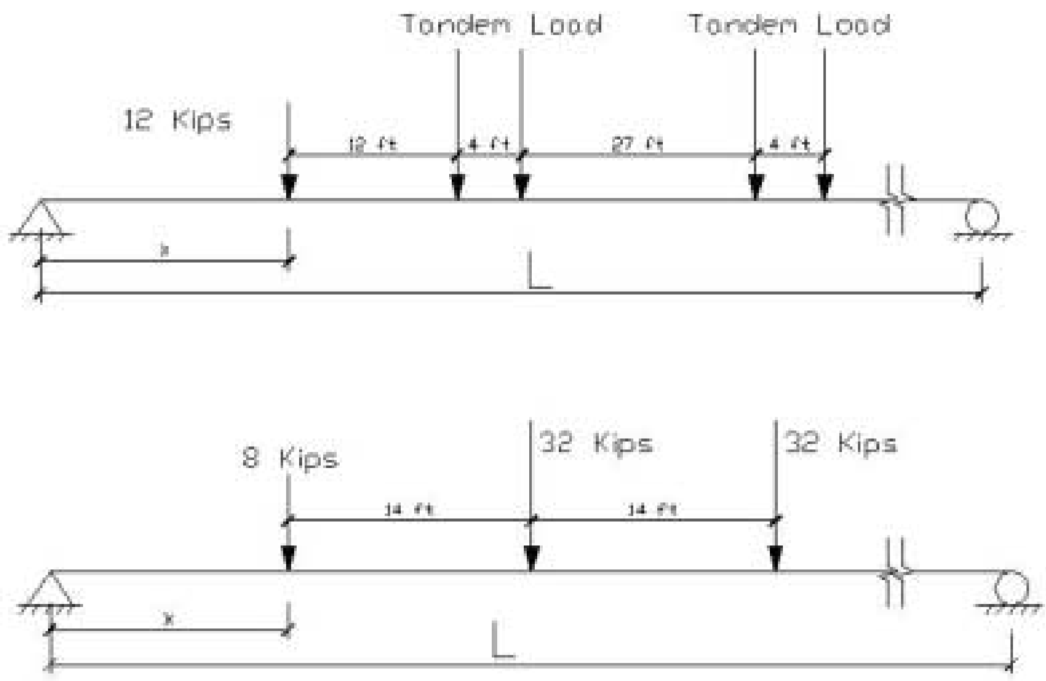

Figure 9
Truck 3S2 and HS20-44 configuration on continuous bridge girder

Determining the critical location of the truck. After generating the influence line for each joint, the position of the truck loads on the bridge girder that would result in maximum positive moment, maximum negative moment, and maximum shear forces was determined. The results are summarized in table 5 of Appendix B.

The maximum moments and shear forces. The maximum positive moment, maximum negative moment, and shear forces due to the wheel loads were calculated by moving the truck loads along the bridge girders in 1 ft. increments. The magnitude of the moment and shear were calculated by taking the sum of the ordinates multiplied by the magnitudes of the loads. Then the loads were placed at the point that produced the maximum value. The location of the truck load that caused the maximum positive moment occurred around 40 percent of the first span, while the location of the maximum negative moment occurred close to the first support of the bridge. The results are presented in Appendix B, tables 6 and 7.

The results of the analysis for the maximum positive moment, the maximum negative moment, and the maximum shear forces for HS20-44 and 3S2 trucks on continuous bridge girders are shown in Appendix B, figures 1 through 3. The increase in the truck load on the moments in the bridge girder was insignificant for girders with spans shorter than 70 ft. However, the impact on the girders with long spans was more significant.

Analysis for Bridge Decks

This subtask focused on the strength and serviceability of bridge decks under the impact of the heavy loads from trucks that are transporting forestry products and Louisiana-produced lignite coal. The evaluation considered composite and non-composite bridge systems. Finite element analysis was used for a typical deck and girder system to determine the effects of the trucks on the stresses in the transverse and longitudinal directions.

All bridges considered for this study had concrete decks. According to the LADOTD Bridge Manual, concrete bridge decks are designed as a continuous span over the girders. The bridge deck analyses for this study were performed using finite element models and GTSTRUDL software. The finite element models for typical bridge decks were generated with a typical 30-ft. bridge-deck width and 8-inch thickness supported by 5 girders. The design load for the bridges included in this study and the loads from 3S2 truck configuration were applied to the deck. Only the “fatigue” load combination, as presented in AASHTO LRFD, was performed for these typical bridge deck models.

The finite element model used for bridge decks in this study simulated the behavior of continuous span bridges. The girders were modeled using Type-IPSL tridimensional elements available in GTSTRUDL. Type-SBCR plate elements were used for the bridge deck. Prismatic space truss members were used to model end diaphragms and the connection between the deck plate elements and the girder elements.

Girder Element Type-IPSL

Properties of tridimensional finite elements were explained in the GTSTRUDL user guide analysis. These were used to model the behavior of general three-dimensional solid bodies. Three translational degrees of freedom in the global X, Y, and Z directions were considered per node. Only force type loads could be applied to these tridimensional elements.

The Type-IPSL tridimensional finite element used was an eight-node element capable of carrying both joint loads and element loads. The joint loads could define concentrated loads or temperature changes, while the element loads could define edge loads, surface loads, or body loads. GTSTRUDL results included the output for stress, strain, and element forces for type-IPSL tridimensional elements at each node. The average stresses and average strains at each node were calculated.

Plate Element Type-SBCR

Properties of plate finite elements were explained in the GTSTRUDL User Guide Analysis. Type plate elements were used to model problems that involved both stretching and bending behavior. The element was a two-dimensional flat plate element commonly used to model thin-walled, curved structures. The type plate finite elements were formulated as a superposition of type plane stress and type plate bending finite elements. For flat plate structures, the stretching and bending behavior was uncoupled, but for structures where the elements did not lie in the same plane, the stretching and bending behavior was coupled.

The type-SBCR plate finite element was a four-node element capable of carrying both joint loads and element loads. The joint loads could define concentrated loads, temperature change loads, or temperature gradients, while the element loads could define surface loads or body loads. GTSTRUDL provided the output for in-plane stresses at the centroid and moment resultants, the shear resultant, and element forces at each node for type-SBCR plate elements. The average stresses, average principal stresses, and average resultants at each node were calculated.

Prismatic Space Truss Members

Properties of space truss members were explained in the GTSTRUDL User Guide Analysis. Space truss members were used when a member experienced only axial forces and where the member was ideally pin connected to each joint. No force or moment loads could be applied to a space truss member. Only constant axial temperature changes or constant initial strain type loads could be applied. The self weight of these members was generated as joint loads which the member was incident upon.

When the prismatic member property option was used, the section properties were assumed to be constant over the entire length of the member. Up to 14 prismatic section properties could be directly specified or stored in tables. If not specified, the values could be assumed according to the material specified. All 14 member cross-section's properties were assumed to relate to the member cross-section's principal axis (local y- and z- axes), which had its origin on the centroidal axis (local x- axis) of the member.

Geometry of Bridge Deck

The geometry of the bridge depended on the width of the roadway, girder type and quantity, number of spans, span length, girder spacing, the bridge skew angle, and the diaphragm skew angle. The span length was measured from the center of one support to the center of an adjacent support. The girder spacing was measured from the center of one girder to the center of an adjacent girder, which was identical and parallel to the previous girder. All the models considered in this study were non-skewed with end diaphragms. The structures analyzed in this study were 30 ft. wide and 3 equal spans. The girders were simply supported and the concrete deck was continuous over the girders. The girders were spaced at 8 ft. in the middle and 7 ft. on the outside. All models contained only five girders, as shown in Appendix C, figures 1 and 2.

Boundary Conditions

The restraints for all models consisted of four joints across the width of the base of the girder at the end and intermediate supports. Also, the two joints that connect the plate elements to the rigid members at the end supports behaved as pins.

AASHTO Loading

A uniform volumetric dead load of 150 pcf was applied to all elements and all members to account for the self weight of the concrete. The truck loading on the bridge was represented by the HS20-44 and 3S2 truck loading with a 1.3 impact factor, based on AASHTO Chapter 3. In addition to the dead and truck loads, a future wearing surface loading of 12psf, according to LADOTD Bridge Manual, was placed on the deck to account for future overlays. The loading conditions used in this investigation were the fatigue loads (self weight, live loads with impact factor) as required by the AASHTO LRFD Bridge Design Manual.

Finite Element Modeling of the Girder over Interior Supports

Since the girders were simply supported and the deck was continuous over the girders, a space would be created between the two girders over the interior supports during the construction of the bridge. Because the end diaphragm did not provide continuity in this case, the girder would require a 2-inch gap between the girders, as shown in Appendix C, figures 3 and 4.

Bridge decks contain longitudinal reinforcing bars for the tensile stresses induced by the negative moment over the support. In construction, the combination of the deck and the bearing pad would restrict the rotation of the girder over the support. Although the girders, when constructed with the end diaphragm, were not joined end to end, the girder was not completely free to act as a truly simply supported beam. In modeling the connection with a two inch gap between adjoining girders, the girders were free to rotate and act as a simply supported beam because the beam was supported by points at the end of the girder and not resting on the pad. Due to the restricted rotation of the girders, tensile and compressive stresses would still exist at the girder ends.

Influence Lines

To determine the critical location of the truck on the bridge, an influence line analysis on the transverse direction was required. The width of the bridge was 30 ft., supported by 5 girders with simple supports. The space between the central 3 girders was 8 ft., and was 7 ft. to the outer girders. Truck loads were placed on the deck as concentrated loads. GTSTRUDL was used to obtain the influence line for each joint of the deck, and Excel was used to analyze the data to get the critical location of the truck, as discussed previously.

Bridge Deck Evaluation

The materials in bridges are subject to high cycle fatigue damage. This means that after many cycles of stresses, even stresses below the maximum permitting stress, enough damage may accumulate to eventually cause the bridge to fail. This would especially occur on those bridges that meet with the heavily traveled vehicles. In this study, the fatigue behavior of three equal span bridges was evaluated. The finite element analysis was performed using GTSTRUDL, and the load combination included the fatigue factor and impact factor to investigate the behavior of the bridge. According to the AASHTO specification, the fatigue factor 0.75 and the impact factor 1.3 were used. The span lengths of the bridges were in the range of 20 to 120 feet with simple support conditions. Truck loads for HS20-44 and 3S2 were applied at critical locations for maximum positive and negative moment in the bridge deck to determine the corresponding stresses. The maximum value of longitudinal, transverse, and shear stresses in the bridge deck were obtained and then grouped as the tensile stress and compressive stress. Appendix C, tables 1 through 4, summarize the results for the maximum stress values of the top and bottom surfaces of the bridge deck, under both HS20-44 and 3S2 truck loads. Also, Appendix C, figures 5 to 10, present the results for stresses on the top surface of the deck, and Appendix C, figures 11 to 16, present the results for stresses on the bottom surface of the deck.

DISCUSSION OF RESULTS

Limitations and Assumptions Affecting Study Results

This study was initiated in July 2004 as a direct result of Louisiana senate resolution 123. The resolution required that a report be prepared by the Louisiana Department of Transportation and Development (DOTD) and submitted to the senate during March 2005. This study included a statewide assessment of the effects of timber, lignite coal and coke fuel transport on Louisiana highways and bridges. Normally, such a study would be conducted over a two-year period. Since only eight months was available for the analysis, report preparation, review of the report and results, and presentation of the study results to the DOTD secretary's office, certain limitations and simplifying assumptions were necessary to complete the work during the time available.

Many of the study's limitations relate to the data that was readily available. Among those limitations related to information about each roadway include:

1. The make up and history of the pavement structure. There are many control sections for which there is no recorded data on the types of layers in the pavement structure the thickness of these layers or when various rehabilitation activities occurred.
2. The values used for the subgrade modulus required in the design of the overlays are likely too large. A single value for resilient modulus of the subgrade soil was used for a whole parish. This single value is thought to be larger than is appropriate for many control sections. The effect of using a soil resilient modulus that is too large is to produce a calculated overlay thickness that is too small. Underestimating the overlay thickness results in overlay costs that are probably too low.
3. The m-values, which reflect the effect of water within the pavement structure, were assumed to be 1.0 since there was no data available on the actual m-values used to design the control sections. The m-values for granular materials used in Louisiana bases are likely to be in the 0.4 to 0.6 range. The effect of using 1.0 instead of the actual m-value is to reduce the overlay thickness and its cost.
4. The traffic volumes included in the latest control section books for each district may be inaccurate. A study currently being conducted for DOTD is addressing this concern. If the average daily traffic (ADT) is too low, then the volume of trucks is too low. Estimates of equivalent single axle loads (ESALs) from the number of trucks will also be too low and the attendant overlay thickness will be too low as will its cost.
5. Estimates of timber tonnage hauled on each of the 39 control sections included in the study were based on estimates of knowledgeable industry personnel and not from actual data taken from mill records. The accuracy of the data

developed by the timber industry is consistent with the level of accuracy of much of the data on the pavement cross sections and ADT data.

6. The fatigue cost for bridges was determined based on average cost for projects completed by DOTD in 2004.

Taking into account all these assumptions and limitations, the project staff believe that the cost data included in this report represent very conservative numbers. If a longer-term, more detailed study were conducted, we believe that the cost data included in this report would be substantially larger. While we believe this to be the case, the cost data included in this report provide the legislature sufficient evidence of the damaging effects from increases in gross vehicle weight (GVW) and the 48,000 lb. maximum tandem axle load to make appropriate changes in the laws governing weights in Louisiana.

Introduction to Pavement Costs

The pavement costs calculated for this study included the costs of overlays required to support the 18-kip ESALs under the various GVW scenarios. The overlay costs were determined for each control section included in the study for both timber and lignite coal.

Coke Fuel

Of the 16 refinery companies in Louisiana, only 3 were involved in transporting coke from the refinery by truck. Of these three, project investigators were able to identify the amounts of coke transported and the destination for only two. Citgo Petroleum in Lake Charles, transported approximately 1,000 tons/month of its coke to a paper mill northeast of Campti. The coke was transported on a 3-S3 vehicle (FHWA class 10 vehicle) with a tandem axle on the tractor and a triple axle on the trailer at a GVW of 88,000 lb. Since an average of only one truck load per day was required to haul the coke fuel, the project staff decided that the pavement and bridge damage would likely be minimal and not significant when compared to that from the other two commodities involved in this study.

The Motiva/Norco Enterprises refinery in Norco transported approximately 48,000 tons of coke in 2003 to the CII Carbon plant in Gramercy. This coke is calcinated by CII Carbon for use in aluminum productions. Since only four truck loads per day are required for transportation, any damage produced will be minimal compared to that produced in transport of other commodities included in this project.

The Conoco-Coke Terminal also transported some coke by truck. However, project staff were unable to secure specific information on the quantity of coke transported or the destinations of the coke.

Based on the above information, the project staff concluded that there was not a large amount of coke fuel transported in Louisiana in 2003 and that further efforts to define amounts and destinations were not justifiable given the deadline for this project. The project staff's efforts needed to be directed toward evaluating the effect of the much larger quantities of timber and lignite coal being transported on Louisiana highways and bridges.

Effects Of Transporting Lignite Coal On Highway Costs

Current Conditions

Lignite coal is produced at two mines in northwest Louisiana in Red River and Desoto parishes at the Dolet Hills and Oxbow mines. The only lignite coal transported on Louisiana highways travels from the Oxbow mine at Armistead, Louisiana

1. Along La 1 for approximately 8 miles to the point where US 84 diverges to head west,
2. The coal then moves approximately 6 miles along US 84, past I49, to La 3248, and
3. Along La 3248 for approximately 2 miles where the trucks turn onto the road to the Dolet Hills power plant.

The lignite coal is hauled in a 3-S3 vehicle (FHWA class 10) with a triple axle on the 42-foot bottom dump trailer which is pulled by a standard truck with a single steering axle and a tandem drive/load axle. The current permitted GVW on this vehicle is 88,000 pounds and Savage Industry, which has the hauling contract, indicated that the tare weight of the truck was 28,000 pounds and that the tandem and triple axles carried about the same weight when loaded to GVW with coal. Under the current conditions (scenario 2) the payload per truck is 60,000 pounds.

Savage Industry personnel indicated that all the lignite coal from the Oxbow mine is transported to the Dolet Hills power plant, and that in 2003, 563,000 tons of coal was hauled. For the 60,000 pound payload per truck, the number of truck loads per day averages 51.42 trucks/day for 365 days a year. Savage Industry also indicated that the amount of lignite mined each year is a fairly stable number, so in this study, a growth rate of zero percent per year has been assumed.

Control Sections Carrying Lignite Coal

Since all the highways carrying lignite coal are in District 04, they were asked to supply pavement cross section and history information. The traffic section in Baton Rouge was asked to supply ADT, vehicle classification data, and traffic growth rates for each of the control sections carrying lignite coal as shown in table 27. However, the traffic section in Baton Rouge indicated that no ADT data was readily available for these control sections, so the district traffic personnel collected traffic count and classification data on these four control sections. The traffic section in Baton Rouge provided estimates for the traffic growth rate.

The pavement cross section data for US 84 was secured using ground penetrating radar data collected for the DOTD in 1995. That data was supplemented with information from District 04 personnel to develop the current cross section. In addition, District 04 personnel provided data on rehabilitation activities on each of the control sections.

With the payload information plus roadway and traffic data, project staff were able to develop the 20-year analysis period cost to rehabilitate these four control sections used to transport lignite coal under three scenarios included in this study:

a. Scenario 1 represents a GVW of 80,000 lbs. and involves no special overweight permits. In that case, it is assumed that a FHWA class 9 vehicle would be used.

b. Scenario 2 represents current conditions, as described above, which include a GVW of 88,000 lbs. being hauled in a FHWA class 10 vehicle, with a triple on the trailer and a tandem on the tractor, each axle carrying the same load.

c. Scenario 3 represents a GVW of 100,000 lbs. carried by the FHWA class 10 vehicle used under scenario 2 with each axle loaded equally.

Table 27
Control section numbers, cross sections, and ADT for roads carrying lignite coal

District No.	Route No.	Control Section No.	ADT	W.C& B. C. Thickness, in.	Base Type & Thickness, in.
4	La 1	53-07	1608	7.0	Soil Cement, 8.5
	US 84	021-04	909	9.5	Soil Cement, 8.5
	US 84	21-03	1122	6.5	Soil Cement, 8.5
	La 3248	816-07	335	5.0	Soil Cement, 8.5

The number of ESALs for each vehicle loaded at the different GVW scenarios were determined from the load equivalence tables in the AASHTO pavement design guide. For US 84, SN of 4.0, P_t of 2.5 in Tables D.4, D.5, and D.6 and the axle loads below produced the following axle load equivalence factors for each scenario:

a. Scenario 1 (80,000 lbs. GVW), axle loads and equivalence factors are:
12,000 lb. steering 34,000 lb. tandem 34,000 lb. tandem
0.213 + 1.11 + 1.11 = 2.433 ESALs/vehicle
NOTE: It was assumed that under the 80,000 lb. GVW the lignite transport vehicle would revert back to FHWA class 9 vehicle instead of the class 10 vehicles.

b. Scenario 2 (88,000 lbs. GVW), axle loads and equivalence factors are:
12,000 lb. steering 38,000 lb. tandem 38,000 lb. triple
0.213 + 1.68 + 0.436 = 2.329 ESALs/vehicle

c. Scenario 3 (100,000 lbs. GVW), axle loads and equivalence factors are:
12,000 lb. steering 44,000 lb. tandem 44,000 lb. triple
0.213 + 2.88 + 0.769 = 3.862 ESALs/vehicle

As seen above, scenario 2 provides the lowest ESALs per truck for the three scenarios. It should be noted that the use of the triple axle on the trailer has a significant positive affect in reducing the destructive effect of axle loads on the pavement. A summary of the factors represented by the three scenarios is provided in table 28 for the control sections on US 84. Similar summary tables for La 1 and La 3248 are provided in tables 29 and 30. La 3248 has a functional classification of collector while US 84 and La 1 are classified as arterials. The data included in table 29 for La 1 used a SN of 3.0 and a P_t of 2.5 so the ESAL calculations used equivalence factors from Tables D.4, D.5 and D.6 of the AASHTO flexible pavement guide. The data included in Table 30 for La 3248 is for a P_t of 2.0 and a SN of 3.0, so the ESAL calculations use equivalence factors from Tables D.1, D.2 and D.3 of the AASHTO flexible pavement guide.

Table 28
Summary of the factors represented by scenarios 1, 2 and 3 for control sections 21-03 and 21-04 on US 84

Scenario	Gross Vehicle Weight, lbs	Payload per vehicle, lbs	ESALs per truck	No. of loads/day required to transport coal in 2003
1	80,000	56,000	2.433	55.09
2	88,000	60,000	2.329	51.42
3	100,000	72,000	3.862	42.85

Table 29
Summary of the factors represented by scenarios 1, 2 and 3 for control section 53-07 on La 1

Scenario	Gross Vehicle Weight, lbs	Payload per vehicle, lbs	ESALs per truck	No. of loads/day required to transport coal in 2003
1	80,000	56,000	2.449	55.09
2	88,000	60,000	2.380	51.42
3	100,000	72,000	4.000	42.85

Table 30
Summary of the factors represented by scenarios 1, 2 and 3 for control section 816-07 on La 3248

Scenario	Gross Vehicle Weight, lbs	Payload per vehicle, lbs	ESALs per truck	No. of loads/day required to transport coal in 2003
1	80,000	56,000	2.349	55.09
2	88,000	60,000	2.309	51.42
3	100,000	72,000	4.129	42.85

Table 31 contains a summary of the traffic growth rate data provided by the traffic section in Baton Rouge and the ADT data collected by District 04 personnel. In addition to this data, District 04 personnel collected classification data on each control section. The classification data was used to predict the number of ESALs applied to each control section under current conditions, scenario 2, for which overlays were designed.

Table 31
Summary of traffic data for control sections used to transport lignite coal

District No.	Route No.	Control Section No.	Length of Control section over which coal is transported		Growth Rate, %/year	ADT from dist. 04
			Centerline miles	Lane miles		
4	La 1	53-07	8.05	16.10	8.7	1,608
	US 84	21-04	2.44	4.88	3.1	909
	US 84	21-03	3.95	9.36	4.9	1,122
	La 3248	816-07	1.55	3.10	10.0	335

Costs Associated with Transporting Lignite Coal

Using the information described above, the overlay costs (in $/12 ft-lane mile) associated with transporting the lignite coal during the 20-year analysis period were calculated for each of the 3 GVW scenarios. These data are tabulated in table 32 and show the time when each overlay was required, the thickness of the overlay, and the cost of the overlay in terms of cost when constructed and in 2003 dollars (net present worth). The data from table 32 was summarized for each GVW scenario in table 33. Notice in table 33 that the scenario 2 minus scenario 1 costs are negative, i.e., by having the coal transported in FHWA class 10 vehicles, the state of Louisiana actually saves money compared to using FHWA class 9 vehicles at a GVW of 80,000 lb. However, when the GVW on the class 10 vehicle increases to 100,000 lb., the required overlay costs on each control section increases.

Table 34 shows the total cost of overlays for the total length of the control sections over which lignite coal is hauled for the analysis period. The total cost data was developed by multiplying the cost per 12-ft. lane mile in table 33 by the number of lane miles on each control section by the lane width of each control section and dividing by 12 feet. This product represents the actual cost of overlays on each control section.

A comparison of the total costs between scenarios 2 and 1 (current conditions and going back to a GVW of 80,00 lbs.) and between scenarios 2 and 3 (current conditions and increasing the GVW to 100,000 lbs.) are shown in the last two columns of table 34. The statewide total for these two scenario comparisons are shown in the last row of table 34. These statewide totals indicate that by allowing lignite coal to be transported in the FHWA class 10 vehicle, with a triple axle on the trailer, Louisiana saves $61,960 during the 20-year analysis period as compared to lowering the GVW back to 80,000 lb. However, if new laws were passed to allow this same vehicle to carry coal at 100,000 lbs., they would cost Louisiana taxpayers an additional $139,670 during the 20-year analysis period, when compared to the current GVW of 88,000 lb. This 20-year period net present worth amounts to $11,210/year for pavement costs. Before developing a recommendation on raising the GVW to 100,000 lb., the bridge costs must be determined and added to the roadway costs.

Bridge Costs on Lignite Coal Control Sections

Table 35 contains the actual calculated cost of damage to 9 bridges that lignite coal trucks cross for scenario 3. The data in this table represents the statewide average cost of fatigue damage to each type of bridges. Notice that the cost is on a per trip basis so the cost per year can be determined by multiplying the number of trips per year by the number of bridges by the cost per trip for each bridge. As noted earlier, lignite coal trucks make 18,767 trips per year carrying the 563,000 tons of coal from the mine at Armistead to the Dolet Hills power plant.

If the legislature were to increase the GVW to 100,000 lb. (scenario 3), the bridge costs incurred by the state on control sections carrying lignite coal would be:

Annual Bridge Cost = 18,767 trips/year* [7 bridges*($5.75/trip) + 2 bridges*($8.9/trip)]
= 18,767 trips/year *[$58.08/ trip]
Annual Bridge Cost = $1,089,420/year

Combined Pavement and Bridge Costs

The total cost of increasing the GVW from 88,000 lbs. to 100,000 lbs. is $1.1 million per year on the lignite coal travel route. Maintaining the current GVW and truck configuration appears to be the most favorable option for Louisiana taxpayers. The authors doubt that Savage Industry could save enough in operating cost by increasing their payload from 60,000 lbs./truck to 72,000 lbs./truck to offset paying $1.1 million dollars per year in additional permit fees to cover fatigue damage done to bridges at the 100,000 lbs. GVW.

Table 32
Overlay thickness, time and cost for highways carrying lignite coal

Highway & Control Section No.	GVW Scenario	Performance Period	Overlay Construction			NPW of Overlay, $/12-ft-lm
			Thick., inches	Year of Overlay	Cost, $/12-ft-lm	
La 1 53-07	1	1	2.00	1997	-	-
		2	3.34	2005	65,170	62,037
		3	3.44	2013	67,292	43,356
						105,363
	2	1	2.00	1997	-	-
		2	3.27	2005	63,852	60,811
		3	3.43	2013	67,069	43,233
						104,040
	3	1	2.00	1997	-	-
		2	3.44	2005	67,133	63,933
		3	3.46	2013	67,630	43,593
						107,526
US 84 21-04	1	1	2.00	1998	-	-
		2	2.53	2005	49,359	45,011
		3	2.50	2013	48,779	30,107
						75,118
	2	1	2.00	1998	-	-
		2	2.44	2006	47,721	43,284
		3	2.46	2014	48,122	29,543
						72,827
	3	1	2.00	1998	-	-
		2	2.63	2005	51,411	47,569
		3	2.54	2013	49,601	31,081
						78,677
US 84 21-03	1	1	2.00	1997	-	-
		2	3.09	2005	60,415	57,510
		3	2.95	2013	57,693	37,171
						94,681
	2	1	2.00	1997	-	-
		2	3.00	2005	58,635	55,843
		3	2.92	2013	57,104	36,810
						92,653
	3	1	2.00	1997	-	-
		2	3.21	2005	62,648	59,636
		3	2.99	2013	58,455	37,662
						97,298
La 3248 816-07	1	1	4.22	2004	82,460	82,460
	2	1	3.95	2004	77,096	77,096
	3	1	4.36	2004	85,261	85,261

Table 33

Net present worth of pavement costs and comparison between scenarios for highways carrying lignite coal

Highway & Control Section	GVW Scenario	NPW of Overlay Costs, $/ 12-ft lane mile	Scenario 2 – Scenario 1 costs, $/12-ft lane mile	Scenario 3 – Scenario 2 costs, $/12-ft lane mile
La 1 53-07	1	105,363	-1,323	3,486
	2	104,040		
	3	107,526		
US 84 21-04	1	75,118	-2,391	5,850
	2	72,827		
	3	78,677		
US 84 21-03	1	94,681	-2,028	4,645
	2	92,653		
	3	97,298		
La 3248 816-07	1	82,460	-5,364	8,165
	2	77,096		
	3	85,261		

Table 34

Statewide total costs between different GVW scenarios for highways carrying lignite coal during the 20-year analysis period

Highway & Control Section	# 12-ft lane miles in control section	TOTAL NPW of Scenario 1 – Scenario 2 Costs	TOTAL NPW of Scenario 3 – Scenario 2 Costs
La 1 53-07	14.758	-19,520	51,440
US 84 21-04	4.473	-10,690	26,160
US 84 21-03	8.823	-17,890	40,980
La 3248 816-07	2.583	-13,860	21,090
Statewide TOTAL		-61,960	139,670

Table 35

Statewide weighted average bridge fatigue costs for trucks hauling lignite coal

Bridge Support Conditions	# of bridges on lignite coal route	Bridge Design Load	Statewide Weighted Cost per loaded trip
Simple	7	HS20-44	$5.75
Continuous	2	HS20-44	$8.9

Developing Statewide Timber Costs from Control Section Data for Each ADT Group

To develop an estimate of the statewide rehabilitation cost for all highways used to transport timber, the cost for all control sections in each category was first developed.

Scenario 2 Net Present Worth of Study Control Sections
The net present worth of the overlay cost for each control section was generated by multiplying the number of lanes x the width of each lane x the control section length x the net present worth of the cost of overlays for the various GVW scenarios. Data used to calculate the net present worth for each control section are shown in tables 36, 37 and 38. In table 36, data in the first four columns were secured from the DOTD control section log books and include the control section number, number of lanes, lane width, and length of the control section or each subsection with a different number of lanes or lane width. Column 5 of table 36 contains the product of columns 2, 3, and 4. Column 6 data represent the scenario 2 net present worth of overlay cost developed from the analysis methodology. Column 7 is the product of columns 5 and 6 and is the net present worth of overlay costs for each control section. Table 36 contains the NPW data for control sections with ADT less than 1,000 vehicles per day, table 37 contains similar data for control sections with ADT between 1,000 and 4,000 vehicles per day, and table 38 contains similar data for control sections with ADT greater than 4,000 vehicles per day.

The net present worth for scenario 2 overlay costs for the 20-year analysis period of each ADT group is summarized in table 39. The data in column 3 of Table 39 are produced by adding together all the totals from column 7 of tables 36, 37, and 38. Similar data for study control sections were developed for scenarios 1 and 3 and are also included in table 39. Notice that the net present worth of overlays is lowest for scenario 1 and greatest for scenario 3. Scenario 2, the present conditions, is between scenario 1 and 3 but is closer to scenario 1 than scenario 3. This result was anticipated because of the non-linear relationship between axle load and equivalence factor used to calculate the ESALs per truck under the different GVW scenarios.

Scenario 2 Statewide Net Present Worth of Overlays for All Control Sections
Data for statewide control sections used for timber transport were tabulated in the same manner as in the first five columns of tables 36, 37 and 38; however, this tabulation of data is not included in the report because of its size. Summaries of the statewide control section dimensions by ADT group are included in table 40. Timber was transported over 504 control sections with ADTs less than 1,000 vehicles per day, 497 control sections with ADTs between 1,000 and 4,000 vehicles per day, and 411 control sections with ADTs greater than 4,000 vehicles per day. The sum of all of the control section dimensions used to transport timber is included in table 40, column 3.

Table 36
Dimensions and scenario 2 overlay cost for 13 study control sections with ADT less than 1,000

Control Section No. (Col. 1)	No. of Lanes (Col. 2)	Lane Width, ft. (Col. 3)	Length, mi. (Col. 4)	Product of (Col. 2* Col.3 * Col.4) (Col. 5)	Scenario 2 NPW of Overlay Cost/ 12-ft-lane-mile (Col. 6)	Scenario 2 Total NPW of Overlay Cost of each Control Section, [((Col. 5)/12)*Col. 6] = (Col. 7), $
45-31	2	10	7.18	143.60	43,347	518,719
88-06	2	10	6.34	126.80	51,659	545,863
89-06	2	10	6.22	124.40	55,495	575,298
128-02	2	10	1.47	29.40	47,899	117,353
130-02	2	10	13.82	276.40	47,015	1,082,912
134-02	2	11	1.00	175.00	85,692	1,249,675
	2	10	7.65			
136-01	2	10	9.21	184.20	91,513	1,404,725
317-05	2	9	3.32	59.76	67,577	336,533
323-01	2	10	7.72	154.40	29,304	377,045
819-19	2	10	5.50	110.00	71,222	652,868
830-01	2	11	8.35	235.90	64,114	1,260,374
	2	10	2.61			
834-08	2	10	9.02	180.40	75,507	1,135,122
863-10	2	9	0.61	10.98	73,873	67,594
TOTALS				**1,811.24**		**9,324,082**

Table 37
Dimensions and scenario 2 overlay cost for 15 study control sections with ADT greater than 1,000 but less than 4,000

Control Section No. (Col. 1)	No. of Lanes (Col. 2)	Lane Width, ft. (Col. 3)	Length, mi. (Col. 4)	Product of (Col. 2* Col.3 * Col.4) (Col. 5)	Scenario 2 NPW of Overlay Cost/ 12-ft-lane-mile (Col. 6)	Scenario 2 Total NPW of Overlay Cost of each Control Section, [((Col. 5)/12)*Col. 6] = (Col. 7)
48-02	2	10	3.57	176.24	40,426	593,723
	2	11	4.22			
	2	12	0.50			
83-01	2	11	6.41	141.02	38,900	457,140
89-03	2	11	17.18	377.96	75,635	2,382,250
190-02	2	10	8.45	169.00	95,864	1,350,085
224-01	2	10	4.47	149.90	91,526	1,143,312
	2	11	2.75			
227-02	2	12	2.61	62.64	66,366	346,431
260-07	2	12	4.94	246.60	102,205	2,100,313
	2	11	5.82			-
263-02	2	11	7.17	157.74	76,984	1,011,955
272-02	2	11	10.79	253.46	36,643	773,961
	2	12	0.67			
415-04	2	10	6.04	120.80	101,037	1,017,106
805-18	2	10	5.52	136.80	68,376	779,486
	4	12	0.44			
	2	12	0.22			
849-26	2	11	2.58	68.76	94,634	542,253
	2	12	0.50			
853-05	2	10	4.07	81.40	65,064	441,351
853-12	2	11	3.56	78.32	77,371	504,975
853-14	2	10	2.81	56.20	18,663	87,405
TOTALS				**2,276.84**		**13,661,519**

Table 38
Dimensions and scenario 2 overlay cost for 11 study control sections with ADT greater than 4,000

Control Section No. (Col. 1)	No. of Lanes (Col. 2)	Lane Width, ft. (Col. 3)	Length, mi. (Col. 4)	Product of (Col. 2* Col.3 * Col.4) (Col. 5)	Scenario 2 NPW of Overlay Cost/ 12-ft-lane-mile (Col. 6)	Scenario 2 Total NPW of Overlay Cost of each Control Section, [((Col. 5)/12)*Col. 6] = (Col. 7)
8-03	4	12	11.29	541.92	39,978	1,805,406
12-13	4	11	0.22	779.12	44,076	2,861,708
	4	12	16.03			
31-07	2	12	2.84	68.12	120,198	682,324
53-09	4	12	8.69	456.72	59,066	2,248,052
	6	12	0.15			
	8	12	0.30			
60-01	4	11	1.86	396.30	49,597	1,637,941
	4	12	6.07			
	6	11	0.35			
67-09	2	12	4.70	112.80	81,708	768,055
67-09	4	11	0.55	161.96	103,390	1,395,420
	4	12	2.87			
92-02	2	10	8.49	169.80	44,653	631,840
253-04	2	11	0.30	130.68	84,819	923,679
	2	12	5.17			
817-31	2	11	2.44	53.68	48,483	216,881
843-09	2	10	0.92	18.40	48,923	75,015
TOTALS				**2,889.50**		**13,246,321**

Table 39
Net present worth of study control section overlay costs for the 20-year analysis period for each GVW scenario by ADT group

ADT Group	NPW of Overlay Costs for each scenario for control sections carrying timber, $		
(Col. 1)	Scenario 1 (80,000 lb GVW) (Col. 2)	Scenario 2 (86,600 lb GVW) (Col. 3)	Scenario 3 (100,000 lb GVW) (Col. 4)
ADT less than 1,000	8,762,810	9,324,082	9,771,175
ADT greater than 1,000 but less than 4,000	13,174,092	13,661,519	14,366,758
ADT greater than 4,000	12,840,257	13,246,321	13,519,451
TOTALS	34,777,159	36,231,922	37,657,384

The scenario 2 statewide cost of overlays for study control sections carrying timber during the 20-year analysis period was calculated using the following equation:

Statewide scenario 2 net present worth =
[(Table 40, Col. 3, Row 1)/(Table 36, Col. 5 Total)] * [Table 36, Col. 7 Total] +
[(Table 40, Col. 3, Row 2)/(Table 37, Col. 5 Total)] * [Table 37, Col. 7 Total] +
[(Table 40, Col. 3, Row 3)/(Table 38, Col. 5 Total)] * [Table 38, Col. 7 Total]

The sum of the above calculation was entered into table 41, column 3, total row as the statewide net present worth of overlay costs for scenario 2.

A similar procedure was used to develop the statewide costs for scenarios 1 and 3. The data showing scenario 1 and 3 overlay costs for each ADT group are contained in tables 42, 43, and 44.

Table 40
Summary of the product of number of lanes x lane width x control section length for each ADT group for 1,412 control sections carrying timber

ADT Group (Col. 1)	**No. of Control Sections (Col. 2)**	**Product of no. lanes X lane width X length for all control sections in each ADT group (Col. 3), (ft of width-miles)**
Row 1: ADT less than 1,000	504	59,423.22
Row 2: ADT greater than 1,000 but less than 4,000	497	80,556.32
Row 3: ADT greater than 4,000	411	82,053.01
TOTAL	**1,412**	**222,032.55**

Table 41
Statewide net present worth of all control section overlay costs for the 20-year analysis period for each GVW scenario by ADT group

ADT Group (Col. 1)	Statewide NPW of Overlay Costs for each scenario for all control sections carrying timber, million $				
	Scenario 1 (80,000 lb. GVW) (Col. 2)	Scenario 2 (86,600 lb. GVW) (Col. 3)	Scenario 3 (100,000 lb. GVW) (Col. 4)	Scenario 2 – Scenario 1 (Col. 5)	Scenario 3 – Scenario 2 (Col. 6)
ADT less than 1,000	287.491	305.905	320.573	18.414	14.668
ADT greater than 1,000 but less than 4,000	466.109	483.355	508.307	17.246	24.952
ADT greater than 4,000	364.624	376.155	383.911	11.531	7.756
TOTAL	**1,118.224**	**1,165.415**	**1,212.791**	**47.191**	**47.376**

Table 42
Scenario 1 and 3 overlay costs for 13 study control sections with ADT less than 1,000

Control Section No. (Col. 1)	Product of # lanes*lane width* length (Col. 2)	Scenario 1 NPW of Overlay Cost/ 12-ft-lane-mile (Col. 3)	Scenario 1 Total NPW of Overlay Cost of each Control Section, [((Col. 2)/12)*Col. 3] = (Col.4)	Scenario 3 NPW of Overlay Cost/ 12-ft-lane-mile (Col. 5)	Scenario 3 Total NPW of Overlay Cost of each Control Section, [((Col. 2)/12)*Col. 5] = (Col. 6)
45-31	143.60	41,698	498,986	47,453	567,854
88-06	126.80	48,751	515,136	58,017	613,046
89-06	124.40	55,300	573,277	55,940	579,911
128-02	29.40	46,886	114,871	50,456	123,617
130-02	276.40	45,115	1,039,149	51,548	1,187,322
134-02	175.00	83,086	1,211,671	91,361	1,332,348
136-01	184.20	90,288	1,013,729	94,818	1,455,456
317-05	59.76	66,041	328,884	71,200	354,576
323-01	154.40	29,304	377,045	29,304	377,045
819-19	110.00	70,458	645,865	73,189	670,899
830-01	235.90	63,687	1,251,980	65,255	1,282,804
834-08	180.40	74,850	1,125,245	76,963	1,157,010
836-10	10.98	73,193	66,972	75,724	69,387
TOTAL	**1,811.24**		**8,762,810**		**9,771,175**

Table 43
Scenario 1 and 3 overlay costs for 15 study control sections with ADT greater than 1,000 but less than 4,000

Control Section No. (Col. 1)	Product of # lanes*lane width*length (Col. 2)	Scenario 1 NPW of Overlay Cost/ 12-ft-lane-mile (Col. 3)	Scenario 1 Total NPW of Overlay Cost of each Control Section, [((Col. 2)/12)*Col. 3] = (Col. 4)	Scenario 3 NPW of Overlay Cost/ 12-ft-lane-mile (Col. 5)	Scenario 3 Total NPW of Overlay Cost of each Control Section, [((Col. 2)/12)*Col. 5] = (Col. 6)
48-02	176.24	40,269	591,417	40,763	598,673
83-01	141.02	37,268	437,961	43,512	511,338
89-03	377.96	71,542	2,253,334	84,844	2,672,303
190-02	169.00	95,034	1,338,395	98,084	1,381,350
224-01	149.90	91,495	1,142,925	91,606	1,144,312
227-02	62.64	66,212	345,627	60,642	347,871
260-07	246.60	99,291	2,040,430	108,962	2,239,169
263-02	157.74	74,020	972,993	83,742	1,100,788
272-02	253.46	35,700	754,044	39,057	824,949
415-04	120.80	98,168	988,224	107,768	1,084,864
805-18	136.80	68,376	779,486	68,376	779,486
849-26	68.76	94,070	539,021	96,148	550,928
853-05	81.40	64,127	434,995	67,452	457,549
853-12	78.32	72,709	474,541	87,517	571,194
853-14	56.20	17,231	80,699	21,776	101,984
TOTAL	**2,276.84**		**13,174,092**		**14,366,758**

Table 44
Scenario 1 and 3 overlay costs for 11 study control sections with ADT greater than 4,000

Control Section No. (Col. 1)	Product of # lanes*lane width*length (Col. 2)	Scenario 1 NPW of Overlay Cost/ 12-ft-lane-mile (Col. 3)	Scenario 1 Total NPW of Overlay Cost of each Control Section, [((Col. 2)/12)*Col. 3] = (Col. 4)	Scenario 3 NPW of Overlay Cost/ 12-ft-lane-mile (Col. 5)	Scenario 3 Total NPW of Overlay Cost of each Control Section, [((Col. 2)/12)*Col. 5] = (Col. 6)
8-03	541.92	39,978	1,805,406	39,978	1,805,406
12-13	779.12	43,904	2,850,540	44,638	2,898,196
31-07	68.12	118,676	673,684	124,188	704,974
53-09	456.72	59,066	2,248,052	59,066	2,248,052
60-01	396.30	49,591	1,637,743	49,615	1,638,535
67-09	112.80	80,376	755,534	87,618	823,609
67-09	161.96	101,706	1,372,692	106,092	1,431,888
92-02	169.80	41,825	591,824	51,246	725,131
253-04	130.68	83,837	912,985	87,373	951,492
817-31	53.68	48,483	216,881	48,483	216,881
843-09	18.40	48,858	74,916	49,100	75,287
TOTAL	**2,889.50**		**12,840,257**		**13,519,451**

Statewide net present worth of overlay costs for all GVW scenarios

Similar calculations were performed to produce the statewide net present worth of overlay costs of all control sections carrying timber during the 20-year analysis period for GVW scenarios 1 and 3. Net present worth costs of all control sections for scenarios 1,2, and 3 are shown in table 41.

Interpretation of Statewide Net Present Worth of Overlay Costs

The data included in table 41 is best interpreted by comparing the costs between the different GVW scenarios. For example, to evaluate the equity of the current permit structure, the difference in cost between scenario 2 and scenario 1, as shown in column 5 of table 41 should be compared to the extra fees paid by the timber industry while transporting timber under scenario 2. However, before making these comparisons, the concept of equity in allocating transportation costs should be discussed.

Cost Allocation Studies

Equity is the concept of allocating pavement costs, produced as a result of the presence of that particular group, to that group through vehicle licensing, registration, permit, fuel and various excise taxes. The Federal Highway Administration last contracted for a cost allocation study in 1997 [4]. That study determined that most commercial vehicles paid their fair share, except for overweight vehicles. Overweight vehicles are those which, permitted or not, are loaded above the 80,000 lb. GVW limit.

From the 1997 cost allocation study, Roberts and Djakfar [2] presented equity ratios for various vehicles, including overweight combination vehicles, as shown in table 45. The overweight vehicles included in this federal cost allocation study are those weighed at various weigh stations operated by the individual states, including permitted overweight vehicles. Notice in table 45, for the two combination truck categories weighing between 80,001 and 100,000 lbs. and greater than 100,000 lbs., the equity ratios are 0.6 and 0.5 respectively. This means that vehicles in these two weight categories are paying only 60 and 50 percent of the costs they incur as a result of their presence. Remember that these data are for all government levels across the U.S. Table 46 provides a breakdown of these data for each government level.

Notice in table 46 that under combination trucks, the row titled “greater than 80,000 lbs.” produces about 10 percent of the cost for all combinations, for all government levels. However, the vehicle-miles traveled carrying these loads represent only 3 percent of the total vehicle miles traveled for all combination trucks [2, page 14]. These data indicate that overweight combination trucks generate a disproportionate cost relative to the taxes they pay to operate on the highway system.

Table 45
Calculated equity ratios for the 1997 federal highway cost allocation study for various vehicle classes for the year 2000 [2]

Vehicle Class / Registered Weight	2000 Forecast Period		
	User Fees Paid[1], %	Cost Incurred[2], %	Equity Ratio
Automobiles	42.6	43.8	1.0
Pickups/vans	21.4	15.4	1.4
All Personal Use Vehicles	64.0	59.2	1.1
Buses	0.1	0.7	0.1
Single Unit Trucks			
Equal to or less than 25,000 lbs.	5.5	3.6	1.5
25,001 to 50,000 lbs.	2.2	3.1	0.7
Greater than 50,000 lbs.	1.8	4.0	0.5
All Single Unit Trucks	9.5	10.7	0.9
Combination Trucks			
Equal to or less than 50,000 lbs.	1.1	0.7	1.6
50,001 to 70,000 lbs.	1.9	1.7	1.1
70,001 to 75,000 lbs.	1.4	1.4	1.0
75,001 to 80,000 lbs.	20.3	22.5	0.9
80,001 to 100,000 lbs.	1.0	1.8	0.6
Greater than 100,000 lbs.	0.7	1.4	0.5
All Combinations	26.4	29.4	0.9
All Trucks	35.9	40.1	0.9
All Vehicles	100.0	100.0	1.0

[1]Percent of total federal user fees paid into the Highway Trust Fund by vehicle class
[2]Percent of total federal cost responsibility incurred by vehicle class

Table 46
Estimated cost responsibility for the year 2000 incurred by vehicle classes for each level of government [2]

Vehicle Class / Registered Weight	Cost Responsibility ($ Millions)			
	Federal	State	Local	Total
Automobiles	12,405	35,988	15,791	64,184
Pickups/vans	4,770	13,678	6,328	24,777
All Personal Use Vehicles	17,396	50,049	22,378	89,832
Buses	221	383	268	871
Single Unit Trucks				
Equal to or less than 25,000 lbs.	1,074	1,755	886	3,715
25,001 to 50,000 lbs.	981	1,867	1,349	4,197
Greater than 50,000 lbs.	1,098	1,929	1,212	4,239
All Single Unit Trucks	3,153	5,551	3,447	12,151
Combination Trucks				
Equal to or less than 50,000 lbs.	222	325	149	696
50,001 to 70,000 lbs.	528	722	306	1,555
70,001 to 75,000 lbs.	408	517	178	1,103
75,001 to 80,000 lbs.	6,329	8,353	2950	17,632
Greater than 80,000 lbs.	778	1,125	450	2,353
All Combinations	8,264	11,042	4,032	23,338
All Trucks	11,417	16,593	7,479	35,490
All Vehicles	28,813	66,642	29,866	125,322

Scenario 2 Pavement and Bridge Costs

In this section, the cost implications of the current permit structure for timber in Louisiana will be interpreted. In 2003, 10,626 harvest permits were issued by DOTD's Truck Permits office. These permits may be purchased for vehicles hauling farm or forest products in their natural state. The DOTD does not differentiate among these permits, so it is not possible to determine exactly how many of these permits were purchased to haul forest products. As a result, the project staff assumed that all the harvest permits were purchased to haul forest products. For a permit fee of $10, a timber truck is allowed to increase the gross vehicle weight from 80,000 lbs. to 86,600 lbs. These two GVWs represent scenario 1 (80,000 lbs.) and scenario 2 (86,600 lbs.) The permit fee income generated for the state of Louisiana is $106,260 which is deposited into the state general fund. Both the Louisiana Forestry Association and the DOTD acknowledge that the $10 fee was initially set to pay for the paperwork associated with issuing the permit and not to pay for road costs that may be associated with the presence of these overweight timber trucks. Therefore, it must be concluded that the Louisiana legislature made a decision to subsidize the timber industry by whatever amount these vehicles cost minus the $106,260 paid for the privilege of hauling at 86,600 lbs.

Data from table 41 provides information on the magnitude of this subsidy to the timber industry. The magnitude of the subsidy is determined by subtracting the net present worth of the total overlay costs of scenario 1 from the net present worth of scenario 2, which in table 41 is the total in Column 5 or $47,191,000 over the 20-year analysis period. This $47,191,000 can be converted to an equivalent annual amount using the following formula:

A = P (A/P, 5%, 20 years)
A = $47,191,000 (0.08024) = $3,786,600/year

The per vehicle subsidy is a minimum of $3,786,600/10,626 or $356/year/truck, minus the $10/year fee currently paid for a total subsidy of $346/year/truck. If only half of the harvest permits are purchased by timber haulers, the subsidy will be $712/year/truck, minus the $10/year fee currently paid for a total subsidy of $702/year/truck. The Louisiana Forestry Association estimates that each log truck pays the equivalent of $835/year in local, state, and federal taxes [5]. Based on the estimates above, it appears that this total tax cost per vehicle should be increased at least $346/year for a total annual tax burden of $1,181/year to cover the pavement costs associated with the presence of these vehicles on Louisiana highways.

In order to get an idea of the magnitude of the bridge costs under scenario 2, the project team selected a small group of bridges and evaluated the effect of the 86,600 lb. GVW on bridge moments, shear, and deflection. The results for bridges of different span lengths are shown in table 47. Spans of 20, 50, 70, 75, and 80 feet were included in this analysis. Columns 2, 3, and 4 of table 47, show the moment, shear, and deflection ratios. As long as the ratio is 1.00 or less, the moment or shear produced by the 86,600 lb. GVW does not result in detrimental effects on the bridge. As can be seen in table 47, none of the moment or shear ratios are greater than 1.00, so no extra bridge cost is produced as a result of fatigue damage to be charged to scenario 2 GVW. So the total cost for scenario 2 is based on pavement costs alone.

Table 47
Moment, shear, and deflection ratios for HS 20-44 and 86,600 lb. GVW loads with equally distributed among the load axles

Span (col1)	**Moment Ratio (col2)**	**Shear Ratio (col3)**	**Deflection Ratio (co4)**
(ft)	*3-S2/HS 20*	*3-S2/HS 20*	*3-S2/HS 20*
20	0.94	0.86	1.07
50	0.81	0.84	0.78
70	0.88	0.96	0.90
75	0.91	0.98	0.93
80	0.93	0.99	0.96

Scenario 3 Pavement and Bridge Costs

Similarly, the overlay cost for increasing the GVW from 86,600 to 100,000 lbs. can be calculated by subtracting column 3 total from the column 4 total in table 41 which results in $47,376,000 over the 20-year analysis period. The annual cost for these overlays is an additional $3,801,000 over the costs for scenario 2. The charge required to recover these costs in annual permit fees would be $358 assuming that all the 10,626 permits were issued to log trucks. The permit fee for scenario 3 is about the same additional cost as for moving from 80,000 lb. to 86,600 lb. GVW. However, for scenario 3 where tandem axle loads approach 48-kips, bridge repair costs contribute a significant additional cost that must be recovered from permit fees if equity is desired. As previously noted, the bridge costs were determined on a per use basis with an average cost of $8.90/trip for the different types of bridges on Louisiana highways when loaded with 48-kip tandem axles.

The magnitude of the costs that a typical log truck operating with 48-kip axles may impose on the bridge system is hypothesized below:

Assume the following scenario:

1. A FHWA type 9 log truck makes 2 trips per day carrying forest products from the forest to the mill.
2. The loaded log truck crosses only 1 bridge on the route to the mill.
3. The trucker works 5 days per week and 40 weeks per year.
4. The total number of trips per year = (2 trips/day)x(5days/wk)x (40 wks/year) = 400 trips/year.
5. If the average bridge repair cost is $8.90/loaded log truck trip, then the annual bridge repair cost for this vehicle would be:
   Bridge Cost =($8.90/trip)X(400 trips/year/truck) = $3,560/year/truck
   NOTE: It is likely that in a typical trip a log truck will cross many more than 1 bridge, in one trip a truck is likely to cross as many as 5 or more bridges.

This example shows that the combined pavement and bridge costs for scenario 3 loads amounts to a minimum total highway cost of $3,906/year/truck. It is unlikely that a timber trucker can afford to pay this annual permit fee. Can the legislature ask the citizens of Louisiana to pay such a high price to allow log truck operators to reduce the number of trips required to haul their products by approximately [((100,000 lb.- 86,600 lb.)/60,000 lb. payload/truck) * 100%], or 22.3 percent by increasing the GVW from 86,600 lb. to 100,000 lb.? It seems unlikely to the project staff that, if asked, the average citizen would be willing to subsidize each timber truck operator $3,560/year so the trucker can reduce the number of trips by 22.3 percent to transport their products.

The Louisiana legislature has the right and the obligation to pass laws that provide favored status for any sector of the economy. However, they are also obligated to ascertain the magnitude of the proposed subsidy and to inform the public of its decision. If the magnitude of the current subsidy, once established, is inappropriate, the legislature is responsible for modifying the weight laws or the permit fee structure to bring all considerations into balance for both the timber industry and the public. One additional factor must be considered before this analysis is complete. Under current statutes, a loaded truck can have one of the load axles weigh up to 48,000 lb. (48-kips) and not be penalized for being overweight. The next section evaluates the effect of permitting one of the axles on a log truck to carry 48-kips. Enforcement personnel on the project review committee noted that they see a very large percentage of timber trucks with trailer axle loads approaching 48-kips. This is done to balance the load that occurs from the long overhang.

Effect of 48,000 lb. Timber Truck Trailer Axles on Pavements for All 3 ADT Groups
In all previous calculations, loads on the truck tandem and trailer axles were assumed to be equally distributed between the axles. However, enforcement personnel on the project technical review committee indicated that many log trucks place extra weight on the trailer axle and often this load is near the 48,000 lb. (48-kips) single axle maximum while the gross vehicle weight is within legal limits. As a result, project staff conducted an additional analysis in which the axle on the trailer was loaded to 48,000 lb. while the steering axle load was 12,000 lb. and the truck tandem carried the difference for the various GVW scenarios. For scenario 1, 80,000 lb. GVW, the steering axle carried 12,000 lb., the truck tandem axle carried 20,000 lb., and the trailer tandem carried 48,000 lb. For scenario 2, 86,600 lb. GVW, the steering axle carried 12,000 lb., the truck tandem axle carried 26,600 lb., and the trailer tandem carried 48,000 lb. For scenario 3, 100,000 lb. GVW, the steering axle carried 12,000 lb., the truck tandem axle carried 40,000 lb., and the trailer tandem carried 48,000 lb.

Project personnel performed calculations using similar procedures to determine overlay costs for each GVW scenario with the 48-kip axle as described in the previous paragraph. The data in tables 48, 49, and 50 were developed to show the net present worth of overlays for each scenario for each of the three ADT groups included in the detailed study of project control sections.

Table 48
Overlay costs produced by 48-kip timber trailer axle for 13 study control sections with ADT less than 1,000 for scenarios 1, 2, and 3

Control Section No. (Col. 1)	Product of # lanes*lane width* length (Col. 2)	NPW of Overlay Cost/ 12-ft-lane-mile (Col. 3)			Total NPW of Overlay Cost of each Control Section, [((Col. 2)/12)*Col. 3] = (Col.4)		
		Scenario 1 (Col. 3a)	Scenario 2 (Col. 3b)	Scenario 3 (Col. 3c)	Scenario 1 (Col. 4a)	Scenario 2 (Col. 4b)	Scenario 3 (Col. 4c)
45-31	143.60	50,225	49,586	50,434	601,026	593,379	603,527
88-06	126.80	58,399	57,640	58,874	617,083	609,063	622,102
89-06	124.40	71,484	71,413	71,493	741,051	740,315	741,144
128-02	29.40	50,456	50,172	50,816	123,617	122,921	124,499
130-02	276.40	51,611	51,100	52,157	1,188,773	1,177,003	1,201,350
134-02	175.00	90,979	90,422	92,063	1,326,777	1,333,237	1,342,585
136-01	184.20	93,517	93,164	94,085	1,435,486	1,430,067	1,444,205
317-05	59.76	71,213	70,822	71,679	354,641	352,694	356,961
323-01	154.40	29,304	29,304	29,304	377,045	377,045	377,045
819-19	110.00	73,216	72,985	73,469	671,147	669,029	673,466
830-01	235.90	65,268	65,130	65,423	1,283,060	1,280,347	1,286,107
834-08	180.40	77,216	77,021	77,212	1,160,814	1,157,883	1,160,754
836-10	10.98	75,911	75,618	76,004	59,536	59,306	59,609
TOTAL	**1,811.24**				**9,940,056**	**9,902,289**	**10,002,354**

Table 49
Overlay costs produced by 48-kip timber trailer axle for 15 study control sections with ADT greater than 1,000 but less than 4,000 for scenarios 1, 2, and 3

Control Section No. (Col. 1)	Product of # lanes*lane width* length (Col. 2)	NPW of Overlay Cost/ 12-ft-lane-mile (Col. 3)			Total NPW of Overlay Cost of each Control Section, [((Col. 2)/12)*Col. 3] = (Col. 4)		
		Scenario 1 (Col. 3a)	Scenario 2 (Col. 3b)	Scenario 3 (Col. 3c)	Scenario 1 (Col. 4a)	Scenario 2 (Col. 4b)	Scenario 3 (Col. 4c)
48-02	176.24	40,446	40,426	40,525	594,017	593,723	595,177
83-01	141.02	40,227	40,844	43,420	472,734	479,985	510,257
89-03	377.96	85,603	84,329	86,047	2,696,209	2,656,082	2,710,194
190-02	169.00	98,133	97,853	98,404	1,382,040	1,378,096	1,385,856
224-01	149.90	91,611	91,597	91,619	1,144,374	1,144,199	1,144,474
227-02	62.64	66,464	66,475	66,519	346,942	347,229	347,229
260-07	246.60	108,983	108,264	109,850	2,239,601	2,224,825	2,257,418
263-02	157.74	83,927	83,146	84,632	1,103,220	1,092,954	1,112,488
272-02	253.46	39,043	38,779	39,398	824,653	819,077	832,151
415-04	120.80	108,434	107,461	108,696	1,091,569	1,081,774	1,094,206
805-18	136.80	68,376	68,376	68,376	779,486	779,486	779,486
849-26	68.76	96,157	95,979	96,370	550,980	549,960	552,200
853-05	81.40	67,510	67,222	67,790	457,943	455,989	459,842
853-12	78.32	88,360	86,971	88,804	576,696	567,631	579,594
853-14	56.20	21,762	21,448	22,159	101,919	100,448	103,778
TOTAL	**2,276.84**				**14,362,383**	**14,271,458**	**14,464,350**

Table 50
Overlay costs produced by 48-kip timber trailer axle for 11 study control sections with ADT greater than 4,000 for scenarios 1, 2, and 3

Control Section No. (Col. 1)	Product of # lanes*lane width* length (Col. 2)	NPW of Overlay Cost/ 12-ft-lane-mile (Col. 3)			Total NPW of Overlay Cost of each Control Section, [((Col. 2)/12)*Col. 3] = (Col. 4)		
		Scenario 1 (Col. 3a)	Scenario 2 (Col. 3b)	Scenario 3 (Col. 3c)	Scenario 1 (Col. 4a)	Scenario 2 (Col. 4b)	Scenario 3 (Col. 4c)
8-03	541.92	39,978	39,978	39,978	1,805,406	1,805,406	1,805,406
12-13	779.12	44,181	44,076	44,248	2,868,525	2,861,708	2,872,875
31-07	68.12	122,286	121,844	122,796	694,177	691,668	697,072
53-09	456.72	68,376	68,376	68,376	2,602,390	2,602,390	2,602,390
60-01	396.30	49,949	49,946	49,952	1,649,566	1,649,467	1,649,665
67-09	112.80	85,023	83,896	85,952	799,216	788,622	807,959
67-09	161.96	104,162	103,972	104,872	1,405,840	1,403,275	1,415,422
92-02	169.80	51,837	50,896	52,139	733,494	720,178	737,767
253-04	130.68	86,938	86,483	87,357	946,755	941,800	951,318
817-31	53.68	50,967	50,967	50,967	227,992	227,992	227,992
843-09	18.40	49,100	49,080	49,128	75,287	75,256	75,330
TOTAL	**2,889.50**				**13,808,648**	**13,767,762**	**13,843,196**

Looking at the total lines in tables 48,49, and 50 and comparing the total NPW cost for each scenario shows that scenario 2 provides the lowest cost for each ADT group. This happens because the number of loads needed to carry the total payload for scenario 1 increases and with the 48-kip axle load, the number of total 18-kip ESALs required to carry the total timber payload for scenario 1 is actually larger than for scenario 2.

Table 51 contains the statewide projected net present worth for each GVW scenario with the 48-kip timber trailer axle. These statewide totals also show that scenario 2 provides the lowest net present worth overlay cost. However, to determine the effect of the 48-kip on statewide net present worth costs, one must compare the statewide totals from table 51 to the statewide net present worth costs for the equally balanced loads from table 41. The combined data from these two tables is shown in table 52 along with the differences between the cost for 48-kip axle loads and the equally balanced loads. This difference for scenario 1 is over $108 million. This is a very significant cost because it represents pavement damage that must be paid for but provides no economic advantage. Allowing such a large imbalance between the axle loads, will cost Louisiana taxpayers over $108 million during the next 20 years. If the legislature were to remove this 48-kip axle allowance and require equally balanced loads on timber trucks, citizens of Louisiana would save over $108 million in the next 20 years, or $8.666 million per year in pavement repair that could be avoided.

Table 51
Statewide net present worth of all control section overlay costs produced by 48-kip timber trailer axle for the 20-year analysis period for each GVW scenario by ADT group

ADT Group	Statewide NPW of Overlay Costs for each scenario for all control sections carrying timber with a 48-kip trailer axle, million $				
(Col. 1)	Scenario 1 (Col. 2)	Scenario 2 (Col. 3)	Scenario 3 (Col. 4)	Scenario 2 – Scenario 1 (Col. 5)	Scenario 3 – Scenario 2 (Col. 6)
ADT less than 1,000	326.114	324.875	328.158	-1.239	3.283
ADT greater than 1,000 but less than 4,000	508.152	504.935	511.760	-3.217	6.825
ADT greater than 4,000	392.124	390.963	393.105	-1.161	2.142
TOTAL	**1,226.390**	**1,220.773**	**1,233.023**	**-5.617**	**12.250**

Table 52
Statewide net present worth of all control section overlay costs for the 20-year analysis period for each GVW scenario by ADT group for equally loaded axles and a 48-kip axle on the trailer

ADT Group (Col. 1)		**Scenario 1 (80,000 lb. GVW) (Col. 2)**	**Scenario 2** (86,600 lb. GVW) **(Col. 3)**	**Scenario 3 (100,000 lb. GVW) (Col. 4)**
ADT less than 1,000	48-kip axle on trailer	326.114	324.875	328.158
	Equally loaded axles	287.491	305.905	320.573
	Difference	38.623	18.970	7.585
ADT greater than 1,000 but less than 4,000	48-kip axle on trailer	508.152	504.935	511.760
	Equally loaded axles	466.109	483.355	508.307
	Difference	42.043	21.580	3.453
ADT greater than 4,000	48-kip axle on trailer	392.124	390.963	393.105
	Equally loaded axles	364.624	376.155	383.911
	Difference	27.500	14.808	9.194
TOTAL	**48-kip axle on trailer**	**1,226.390**	**1,220.773**	**1,233.023**
	Equally loaded axles	**1,118.224**	**1,165.415**	**1,212.791**
	Difference	**108.166**	**55.358**	**20.232**

Allowing timber trucks to be loaded with a 48-kip axle on the trailer under scenario 2 costs taxpayers and extra $55 million in pavement costs over the next 20 years. Under scenario 3, the extra pavement costs amount to over $20 million when a 48-kip axle load is permitted on the timber truck trailer. Clearly, permitting one of the truck axles to be loaded to 48-kips is very costly for taxpayers of Louisiana. It is in the best interests of the taxpayers of Louisiana for the legislature to require that the loads on timber trucks be more equally distributed between the truck axle and the trailer axle. Enforcing equally loaded axles can only be accomplished by eliminating the weight provision that allows individual axle loads up to 48-kips.

Combined Effect of GVW and 48-kip Axle Loads on Pavements and Bridges

If the Louisiana legislature decides to permit GVWs above the 80,000 lb. and at the same time allow individual axle loads of up to 48-kips, they must consider the cost produced by the combined effects and charge permit fees are proportional to the repair cost for the damage produced. For example, there are six combinations of GVW with equally loaded axles and 48-kip axle loads for which permit fees should be calculated to recover costs incurred by the presence of these various GVWs and axles:

1. 80,000 lb. GVW but with a permit to have a 48-kip axle
   The permit fee per vehicle per year would include:
   a. Cost of the 48-kip axle on highway costs
      {[1,226.390 – 1,118.224]million}* 0.08024/ 10,626=$817/yr/truck
   b. Cost of the 48-kip axle on bridge repair cost
      $3,560/yr/truck
   c. TOTAL EXTRA COST = $4,377
2. 80,000 lb. GVW but with balanced axles
   The permit fee per vehicle per year would include:
   a. Cost of the balanced axle on highway costs
      $0.00
   b. Cost for bridge repair
      $0.00
   c. TOTAL EXTRA COST = $0.0
3. 86,600 lb. GVW but with a permit to have a 48-kip axle
   The permit fee per vehicle per year would include:
   a. Pavement cost for 86,600 lb. GVW with balanced loads
      $346/yr/truck
   b. Pavement cost for the 48-kip axle at 86,600 lb.
      {[1,220.773 - 1,165.415] million}*0.08024/10,626 = $418/yr/truck
   c. Cost for bridge repair
      $3,560/yr/truck
   d. TOTAL EXTRA COST = $4,324/yr/truck
4. 86,600 lb. GVW but with balanced axles
   The permit fee per vehicle per year would include:
   a. Pavement Cost for 86,600 lb. GVW with balanced axles
      $346/yr/truck
   b. Cost of bridge repair
      $0.00
   c. TOTAL EXTRA COST = $346/yr/truck

5. 100,000 lb. GVW but with a permit to have a 48-kip axle
   The permit fee per vehicle per year would include:
   a. Pavement cost for 86,600 lb. GVW with balanced axle loads
      $346/yr/truck
   b. Additional pavement cost from 86,600 lb. GVW to 100,000 lb. GVW with balanced axles
      $358/yr/truck
   c. Pavement Cost for the 48-kip axle at 100,000 lb. GVW
      {[1,233.023-1,212.791] million}*0.08024/10,626 = $153/yr/truck
   d. Cost of bridge repair
      $3,560/yr/truck
   e. TOTAL EXTRA COST = $4,417/yr/truck
6. 100,000 lb. GVW but with balanced axles
   The permit fee per vehicle per year would include:
   a. Pavement cost for 86,600 lb. GVW with balanced axle loads
      $346/yr/truck
   b. Additional pavement Cost in moving from 86,600 lb. GVW to 100,000 lb. GVW with balanced axles
      $358/yr/truck
   c. Cost of bridge repair
      $3,560/yr/truck
   d. TOTAL EXTRA COST = $4,264/yr/truck

These data are summarized in table 53.

Table 53
Summary of extra pavement and bridge costs for GVW scenarios and axle load conditions

GVW Scenario	Axle Loading	Extra Pavement Cost, $/yr/truck	Extra Bridge Cost, minimum estimate, $/yr/truck	TOTAL EXTRA COST, $/yr/truck
Scenario 1 @ 80,000 lb.	Equally loaded axles[1]	0	0	0
	48-kip axle	817	3560	4,377
Scenario 2 @ 86,600 lb.	Equally loaded axles	346	0	346
	48-kip axle	346 + 418	3560	4,324
Scenario 3 @ 100,000 lb.	Equally loaded axles	346 + 358	3560	4,264
	48-kip axle	346 + 358 + 153	3560	4,417

[1]Equally loaded axles occur when the truck rear axle and trailer axle weights are equal

It appears obvious that, in light of the data in table 53, the legislature should immediately act to remove the provision allowing a single axle load of 48-kips. Removing this provision would prevent the citizens of Louisiana from having to accrue the obligation to pay at least $3,560/yr/log truck in bridge damage costs plus another $418/yr/log truck in the pavement damage costs. It is probable that the cost of damage per truck per year is much larger that this total of $3,978 because the bridge damage cost was based on a very conservative estimate of a log truck crossing only one bridge per trip and making only two trips per day. The potential minimum cost of pavement and bridge damage per year is $3,978/yr/truck times 10,262 permits/ year for a total annual damage cost of $40.822 million. This annual damage cost of $40.822 million is occurring under current laws with the burden for payment being borne by the taxpayers of Louisiana; however, the payments for damage are not being paid, but the damage is accumulating as a backlog of deferred maintenance. This backlog of bridge damage continues to accumulate and will have to be paid in due time. These damaged bridges will either have to be repaired or the citizens of Louisiana will be driving on unsafe bridges. To pay for the damage produced by these 48-kip axle loads, gas taxes must be raised or permit fees increased. The most logical action to counter the continued damage produced by these 48-kip axle loads is to rewrite the statutes and rescind the 48-kip axle load provision.

In the event that the legislature decides to rescind the 48-kip axle load provision and also decides that the scenario 2 subsidy of $47,191,000 over the next 20 years is too large, some of the available options are:

1. Modify the law to eliminate this overweight permit, i.e., revert to scenario 1, a GVW of 80,000 lbs. Remembering the data in table 53, it is imperative that the legislature rescind both the 48-kip single axle load and the 86,600 GVW for this option to be feasible.
2. Increase the permit fee to a level more consistent with the pavement costs produced by vehicles hauling timber at 86,600 lb. GVW with equally loaded axles. The data in table 53 indicates that this annual fee should be at least $346/yr/truck assuming all 10,626 harvest permits are log trucks. Since it is impossible for all these permits to be log trucks, the number of log truck permits should be determined and the annual pavement cost divided by the actual number of permits issued annually.

   <u>Author's note</u>: *Equity demands that any change in permit fees be deposited into the highway trust fund to finance a portion of the extra cost, paid by DOTD, to more frequently rehabilitate the roads over which overweight permitted timber trucks travel.*

In exploring option 2 above, the Louisiana legislature must decide if it desires to provide a subsidy to the timber industry. Historically, Louisiana has subsidized agricultural industries by allowing them special privileges in the transportation of their products. Since many of these products are perishable, they need to either get to the market or to the first processing point as quickly as possible. In the case of timber, this reasoning is specious since the product is not perishable in the time frame typically applied to agricultural products. As a result, the legislature should consider if an authentic need for timber overweight permits exists. In addition, the legislature must also consider the magnitude of the increased transportation cost to the timber industry if the payload is decreased by 6,600 lb. per truck load to a GVW level of 80,000 lb. The payload per truck under the 86,600 GVW is 60,000 lb. of timber. Decreasing the payload by 6,600 lb. per truck would generate one additional truck trip for every 9.09 loads under the current law. As a result, the operating cost for a trucker would increase by 11 percent if the GVW is reduced from 86,600 lbs. to 80,000 lbs. However, the legislature must also consider the increase in overlay costs paid by the DOTD which amount to: {[Scenario 2 total cost – Scenario 1 Cost] / Scenario 1 total cost } * 100% = 1.96 percent when loads are equally distributed on the axles.

So the legislature must answer this question: "Should each citizen of Louisiana pay 1.96 percent more for highway costs in order to reduce timber truck operating costs by 11 percent?" The answer is especially important when considering the fact that by subsidizing timber truck operating costs, the citizens of Louisiana are lowering the cost of lumber for citizens of other states who purchase Louisiana timber. The legislature may also consider placing a user tax fee on the timber owner for use of the highway system to transport the timber from the harvest point to the processing site. Such a charge should be on the basis of weight and distance the timber travels on the road system. The user tax fee could be subtracted from the purchase price of the timber at the first processing plant, when the timber is weighed. Such a tax would be applied directly to the party benefiting from having the road system available for hauling the timber to the first processing point.

The bridge costs presented earlier show that if the GVW is reduced from 86,600 lbs. to 80,000 lbs. and the axle loads are equal, the damage cost to repair and reconstruct bridges will likely be reduced to nearly zero. In the bridge analysis the only load for which costs were estimated was for FHWA type 9 trucks with maximum tandem load of 48, 000 lbs., which corresponds most nearly to scenario 3. The extra bridge fatigue costs produced by trucks at 80,000 lb. GVW with equally loaded axles were estimated to be zero in an independent research study conducted by staff of the Louisiana Transportation Research Center [6]. However, if the 48-kip axle load provision is not eliminated by the Louisiana legislature, the cost of bridge damage for all GVW scenarios is at least $3,560/yr/truck.

Introduction to Bridge Costs

Previous Studies

The truck industry is faced with the demand of increasing the truck weight to get more carrying capacity [8, 9, 10, 11]. On the other hand, bridge owners in the United States can control the loading on the bridges to limit the deterioration of the existing bridge infrastructure to keep the structure in a safe condition. To solve this problem, regulations allow the truck weights to increase to a certain range while guaranteeing the safety and serviceability of the bridge systems. The Federal legislation known as Federal-aid Highway Act introduced a program regulating truck weights. This legislation restricts the gross weights of trucks and weights of different axles and axle groups. The maximum gross weight of the vehicle is 80,000 lbs., while the limit for the single axle load is 20,000 lbs. and 34,000 lbs. for the tandem axles. The axle group weights are regulated based on the truck weight formula, also known as "Formula B," given by:

$$W = \frac{BN}{2(N-1)} + 6N + 18$$

Where W is the overall gross weight (in pounds), B is the length of the axle group (in feet), and N is the number of axles in the axle group. By using the Formula B, the overstressing of the bridges with an HS20 design load can be avoided by more than 5 percent and the bridges with an H15 design load can be avoided by more than 30 percent.

This formula is based on the principle that overstressing H15 bridges by 30 percent is still acceptable for bridge safety and serviceability. Most of the H15 bridges are built on low heavy-truck volume highways while the HS20-44 bridges are usually built on interstate highways. This means that if the bridge is overstressed by more than 5 percent, a high risk exists. However, engineering experiences in some states and the province of Ontario show that the results of Formula B are very conservative. Many states have increased their legal loads above the standard. For example, Minnesota allows a winter increase in GVW of 10 percent during dates set by the transportation commissioner based on a freezing index. Michigan allows loads up to 154,000 lbs., and most western states allow loads up to 131,000 lbs.

The Federal Highway Administration (FHWA) supported research to develop another truck weight formula known as the TTI formula, which is based on the same overstressing criterion as the Formula B. Compared to the Formula B, the TTI formula allows higher weights for shorter vehicles, tandem, and tridem axle groups, but it allows smaller gross weights than Formula B for longer vehicles. The TTI formula is given by:

$W=34 + B$ (Kips) for $B < 56$ ft.
$W=62 + 0.5B$ (Kips) for $B >56$ ft.

In 1990, the Transportation Research Broad (TRB) finished research on a modification of the TTI formula, which reduced the limits on axle loads and allowed the higher gross weights. However, the modified TTI formula established stress limits on the bridges whose design load is the HS20 truck load without consideration of the H15 truck load; the modified formula is given by:

W = 26 + 2.0B (Kips) for B < 23 ft.
W = 62 + 0.5B (Kips) for B > 23 ft.

Short Term Effects on Simple and Continuous Span Bridges

Simple span bridges. In this study, simple span bridges were grouped based on their design load H15 or HS20-44 AASHTO trucks. The effects of 3S2 truck loads on these bridges were investigated by comparing the flexural, shear, and serviceability conditions.

Each of the truck loads was placed on the bridge girder with simple supports and spans from 20 to 120 feet. Absolute maximum moment, shear, and deflection of the bridge girder were calculated under each load configuration and the results are listed in Appendix B, tables 2 through 4. The critical conditions for each bridge were determined based on AASHTO Chapter 3 using both lane load (0.48 kips/ft.) and truck loads. In this study, the HS20-44 and 3S2 truck loads controlled the critical conditions. For H15 truck loads, the governing load conditions are summarized in Appendix B, table 8.

The effects of 3S2 trucks loads on bridges designed for H15 and HS20-44 truck loads were evaluated by normalizing the critical conditions for each bridge span to the design load. The results are presented in Appendix B, table 9.

Effects of 3S2 trucks on simple span bridges with H15 design loads. The effects of 3S2 truck loads on simple span bridges designed for H15 truck loads are presented in Appendix B, figure 4. The ratio of the absolute maximum moment varied between 1.62 and 2.07. The ratio of the shear forces varied between 1.77 and 2.10. These high ratios could induce flexural and shear cracks in the bridge girders. Moreover, these ratios were much higher than the margin of safety (30 percent) available for bridges designed for H15 truck loads. Previous studies [9, 10] reported that due to changes in the design codes and practices, a margin of safety of about 30 percent existed in bridges designed for H15 truck loads.

The ratio for deflection caused by 3S2 truck loads as compared to H15 truck loads varied between 1.83 and 3.18. Deflection is a serviceability criterion, and high ratios as reported in this study, would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the 3S2 trucks. Also, the high ratios obtained in this study could result in more cracking in the bridge girders and bridge decks. Such cracks would require additional inspections as well as early and frequent maintenance.

Effects of 3S2 trucks on simple span bridges with HS20-44 design loads. The effects of 3S2 truck loads on simple span bridges designed for HS20-44 truck loads are presented in Appendix B, figure 5. The ratio of the absolute maximum moment varied between 0.98 and 1.29. The ratio of the shear forces varied between 0.97 and 1.34. Where the bridge span was similar to the length of the 3S2 truck, the ratios of the absolute maximum moment and shear were within 10 percent. This confirms the findings in the previous studies that focused on bridge formula. The studies increased the GVW and the truck length to minimize the impact on the stresses in the bridge girders. However, bridge girders with absolute maximum moment ratio or shear larger than 1.1 would be overstressed.

The ratio for deflection caused by 3S2 truck loads as compared to HS20-44 truck loads varied between 0.94 and 1.42. The above discussion on the ratio of the absolute moment was applied to the ratio of deflection. Deflection was a serviceability criterion and high ratios as reported in this study would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the 3S2 trucks.

The bridges in this study with absolute maximum moment ratios and shear ratios that were greater than 1.1 could experience more cracking in the bridge girders and bridge decks. Such cracks would require additional inspections along with early and frequent maintenance.

Effects of 3S2 trucks on continuous span bridges with HS20-44 design loads. The effects of 3S2 truck loads on continuous span bridges designed for HS20-44 truck loads were presented in Appendix B, table 10, figures 6 and 7. The ratio of the maximum positive moment varied between 1.0 and 1.28. For the maximum negative moment the ratio varied between 1.0 and 1.48. The ratio of the shear forces varied between 0.98 and 1.40. Where the bridge span was similar to the length of the 3S2 truck, the ratio of the maximum positive moment and shear forces were within 10 percent. This confirmed the findings in the previous studies that focused on bridge formula. The previous studies increased the GVW and minimized the impact on the stresses in the bridge girders by increasing the truck length. However, bridge girders with a maximum positive moment ratio or shear larger than 1.10 would be overstressed.

The ratio for negative moment for spans between 105 ft. to 130 ft. was around 1.4. The high ratios for the negative moment would increase the compressive stress in the bridge decks. These conditions could result in compression cracks in bridge decks. The bridges in this study with ratios that were greater than 1.1 could experience more cracking in the bridge girders and bridge decks. Such cracks would require additional inspections along with early and frequent maintenance.

Short Term Effects of Hauling Timber and Lignite Coal on Louisiana Bridges

Tables 1, 2 and 3 of Appendix A provide the bridges and their categories that were analyzed in this study. The discussion on state bridges is presented first, followed by parish bridges.

Posted bridges and design load low categories. This study, included 169 posted bridges and 55 bridges with low design load. It is recommended that the 3S2 trucks with truck configuration similar to those considered in this study not be allowed to cross the bridges that are in the posted or design load low categories.

Simply supported bridges with design load H15. The effects of 3S2 truck loads on simple span bridges that were designed for H15 truck loads are presented in Appendix B, table 11. The span for most of these bridges is 20 ft. The ratio of the absolute maximum moment and shear due to 3S2 and H15 truck loads were 1.62 and 1.77 respectively. These high ratios could induce flexural and shear cracks in the bridge girders. Moreover, these ratios were much higher than the margin of safety available for bridges designed for H15 truck loads. Previous studies reported that changes in the design codes and design practices could cause a 30 percent margin of safety in bridges designed for H15 truck loads [9,10].

The ratio for deflection caused by 3S2 truck loads as compared to H15 truck loads varied between 1.83 and 2.73. Deflection was a serviceability criterion and high ratios, as reported in this study, would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the 3S2 trucks. Also, the high ratios obtained in this study could result in more cracking in the bridge girders and bridge decks. Such cracks will require additional inspections along with early and frequent maintenance.

Simply supported bridges with design load HS20-44. The effects of 3S2 truck loads on simple span bridges designed for HS20-44 truck loads are presented in Appendix B, table 12. The span for most of these bridges is 20 ft., the ratio of the absolute maximum moment and shear due to 3S2 and HS20-44 truck loads are 1.22 and 1.1 respectively. Previous studies reported that changes in the design codes and design practices could cause a margin of safety of about 5 percent to 10 percent in bridges designed for HS20-44 truck loads [9,10].

This study included 60 bridges with span lengths between 40 ft. and 66 ft. The ratio for the absolute maximum moment was within the margin of safety. There were 57 bridges with span lengths between 70 ft. and 120 ft., and 38 bridges with span lengths between 25 ft. and 35 ft. The ratio for the absolute maximum moment was larger than 1.1, or more than the 10 percent margin of safety. Therefore, the bridges in Appendix B, table 12 with ratios that are higher than the margin of safety for bridges designed for HS20-44 truck loads could experience flexural and shear cracks in the bridge girders and bridge decks. Such cracks would require additional inspections along with early and frequent maintenance.

The ratio for deflection caused by 3S2 truck loads, as compared to HS20-44 truck loads, varied between 0.94 and 1.42. Deflection was a serviceability criterion and the bridges with high ratios would result in uncomfortable riding conditions for vehicles crossing the bridges at the same time as the 3S2 trucks. Also, high ratios could result in higher cracking in the bridge girders and bridge decks. Such cracks would require additional inspections and could result in early and frequent maintenance.

The results in Appendix B table 12, indicated that as the bridge span increased beyond 70 ft. all the ratios increased, especially the ratio for the absolute maximum moment. Consequently the flexural stresses will increase. The methodology for this study was simplified, with the approval of the Project Review Committee, in order to meet the legislative due dates. The results of this study were limited to bridges with span lengths of 120 ft. Therefore, the 13 bridges in this study with span lengths longer than 120 ft. were considered outliers, and should be evaluated with more details under a separate study.

Continuous bridges with design load HS20-44. The effects of 3S2 truck loads on continuous bridges designed for HS20-44 truck loads are presented in Appendix B, table 13. The ratio of the maximum positive moment due to 3S2 and HS20-44 truck loads varied between 1.00 and 1.29. The ratio of the shear forces varied between 0.98 and 1.40. Previous studies reported that changes in the design codes and design practices could cause a margin of safety of about 5 percent to 10 percent in bridges designed for HS20-44 truck loads [10].

This study included 42 bridges with span lengths between 40 ft. and 70 ft. The ratios for the maximum moment were within the margin of safety. There were 3 bridges with span length equal to 20 ft., and 81 bridges with span lengths between 70 ft and 130 ft., for which the ratio for the maximum positive moment was larger than 1.1, or more than the 10 percent margin of safety. Therefore, these bridges could experience flexural and shear cracks in the bridge girders and bridge decks. Such cracks would require additional inspections along with early and frequent maintenance.

The ratio for the maximum negative moment was higher than the margin of safety, except for the 3 bridges with span lengths equal to 20 ft. The high values in negative moment would result in high compressive stresses in the bridge decks. Such conditions could result in an increase in the compression cracks and would require additional inspections and could result in early and frequent maintenance.

The methodology for this study was simplified, with the approval of the Project Review Committee, in order to meet the legislative due dates. The results of this study were limited to continuous bridges with span lengths of 130 ft. Therefore, the 19 bridges which were part of this study with span lengths longer than 130 ft. were considered outliers. Also considered outliers were 2 continuous bridges designed for H15 truck loads with span lengths of 70 ft. and 200 ft. All bridges in the outlier category should be evaluated for more details under a separate study to determine the effects of hauling timber and lignite coal on these bridges.

Similar analyses were performed for parish bridges, and the results are presented in Appendix D.

Bridge Decks

This part of the research focused on the strength and serviceability of bridge decks due to the impact of the heavy loads from the trucks that are transporting forestry products, Louisiana-produced lignite coal, and coke fuel. Finite element analysis was used for a typical deck and girder system to determine the effects of the trucks on the stresses in the transverse and longitudinal directions, and the shear stress.

Continuous bridge decks. The effects of 3S2 truck loads on continuous bridge decks designed for HS20-44 truck loads were presented in Appendix C, tables 5 and 6 and figures 17 to 22. The stresses were computed separately at the top and bottom surfaces. The ratios of the maximum stresses at the surface were grouped based on whether they were tensile or compressive stresses.

At the top surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction varied between 0.91 and 1.74 and between 0.71 and 1.37 in the transverse direction. The ratio of shear stress varied between 0.87 and 1.59. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction varied between 0.58 and 1.09, and between 0.90 and 1.10 in the transverse direction; the ratio of shear stress varied between 0.98 and 2.23. The ratio of maximum compressive stress was mostly smaller than the ratio of maximum tensile stress. The ratios of maximum tensile stress in the longitudinal direction were larger than 1.15 when the span length was longer than 30 ft. Therefore, these bridge decks may experience cracks in the longitudinal direction. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or 3S2 truck loads may differ from each other. The difference is what makes the ratio of 3S2 to HS20-44 truck for some span lengths less than 1.

At the bottom surface of the bridge deck, the ratio of maximum tensile stress in the longitudinal direction varied between 0.58 and 1.09, in the transverse direction varied between 0.90 and 1.10, the ratio of shear stress varied between 0.98 and 2.23. For the ratio of maximum compressive stress, the ratio of maximum stress in the longitudinal direction varied between 0.91 and 1.74, in the transverse direction varied between 0.71 and 1.37; the ratio of shear stress varied between 0.87 and 1.59. The ratio of maximum tensile stress was mostly smaller than the ratio of maximum compressive stress. The ratios of maximum compressive stress in the longitudinal direction were larger than 1.15 when the span length was longer than 30 ft. Therefore, these bridge decks may experience cracks in the longitudinal direction. Such cracks would require additional inspections along with early and frequent maintenance. The locations of maximum stresses due to HS20-44 or 3S2 truck loads may differ from each other.

The results show that the ratio of tensile stresses at the top surface is of the same magnitude as the ratio of compressive stresses at the bottom surface. Also, the ratio of compressive stresses at the top surface is of the same magnitude as the ratio of tensile stresses at the bottom surface. These similarities confirm that the bridge deck is under a stable stress state, whether the stresses are in the tension zone or the compression zone.

We should consider that the locations of maximum stresses due to HS20-44 or 3S2 truck loads may differ. Further research should be applied to obtain the ratios of the stresses at same location to evaluate the deck behavior under heavy truck loads.

Long Term Effects on Simple and Continuous Span Bridges

The long term effects of heavy trucks, such as trucks hauling timber and lignite coal on bridges and bridge decks, play an important role in the bridge life evaluation. The selected bridges for this study were designed under AASHTO standard H15 or HS20-44 truck loads. Overloaded trucks traveling across these bridges will increase the cost of maintenance and rehabilitation. An accurate estimate for the cost of the damage is hard to obtain since fatigue damage may lead to many actions including repairs, testing, rehabilitations, and replacements.

Many studies were done and methods used to evaluate the remaining lives of bridge structures. These studies were sponsored by federal committees such as AASHTO and NCHRP and by State DOTs. The use of these methods in this study is hindered by the amount of data on timber and lignite trucks needed. The site-specific information available for this study on timber and lignite trucks was very limited and statistically insufficient for use with the NCHRP 495 approach or the other methodologies discussed above. The approach used in this study was discussed and approved during the PRC meeting on September 2, 2004. A similar method was used in the study prepared for OHIO DOT.

Fatigue is an important performance criterion for bridges that are evaluated. Most of the bridges in Louisiana are designed for 50 years fatigue life. Overloaded trucks will definitely shorten the life of the bridges. The bridges in this study were evaluated for fatigue cost based on the flexural and shear results of the analyses performed in previous tasks of this study. The truck ADT value of 2,500 was used based on a review of the bridges considered in this study and the recommendations of AASHTO. The bridge costs used in this study were based on projects completed by DOTD during 2004. The average cost to replace a state bridge was $3M and parish bridge was $140 per square foot. The average cost to replace concrete bridge girder and bridge deck was $90 per square foot.

The following equation was used to determine the percentage of the life of the bridge used when a truck crosses it.

$$\% \text{ of life} = \frac{(\text{Ratio from analysis})^3}{(2500 \text{ trucks per day} * 365 \text{ days per year} * 50 \text{ years})} * 100$$

The estimated cost per trip across the bridge was obtained by multiplying the percentage of the life of the bridge by the total cost of the bridge. In this study, the cost to replace concrete bridge girder and deck was considered to be $90 per square foot.

*Cost per Trip on State or Parish Bridge = (% of life) * (*$90 per square foot*)*

The effect of the trucks hauling timber and lignite on the fatigue life of the bridge was ignored when the "ratio from analysis" was equal to or less than one. Therefore, the cost per trip for fatigue calculation is zero.

Since the trucks are operating on a broad route structure, the total damage cost was estimated on a per bridge basis. This applied to cases with no defined route for the vehicle. The weighted average over all spans lengths and number of spans was used.

Long Term Effects of Hauling Timber and Lignite Coal on Louisiana Bridges

The long term effects of 3S2 trucks hauling timber and lignite coal with maximum tandem load of 48,000 lbs. and steering axle of 12,000 lbs. on Louisiana state and parish bridges were determined and summarized in table 54.

Table 54
Fatigue Cost for 3S2 trucks on Louisiana bridges.

Bridge Support Condition	Design Load	Cost per Trip	
		State Bridge	**Parish Bridge**
Simple	H15	$8.5	NA
Simple	HS20-44	$5.75	$1.05
Continuous	HS20-44	$8.9	NA

State Bridges

Simply supported bridges with design load H15. The long term effects of 3S2 trucks hauling timber and lignite coal on Louisiana state bridges were calculated based on flexural and shear analyses performed in previous tasks. The span for most of these bridges was 20 ft. and the controlling factor was the high ratio of shear forces. The results are presented in Appendix E, table 6. The estimated fatigue cost per trip is $8.50.

Simply supported bridges with design load HS20-44. The long term effects of 3S2 trucks hauling timber and lignite coal on Louisiana state bridges were calculated based on flexural and shear analyses performed in previous tasks. The span for most of these bridges was 20 ft.

and the controlling factor was the high ratio of flexural moments. The results are presented in Appendix E, table 7. The estimated fatigue cost per trip is $5.75.

Continuous bridges with design load HS20-44. The long term effects of 3S2 trucks hauling timber and lignite coal on Louisiana state bridges were calculated based on flexural and shear analyses performed in previous tasks. The results are presented in Appendix E, tables 8 and 9. The estimated fatigue cost per trip is $8.90.

Parish Bridges

The long term effects of 3S2 trucks hauling timber and lignite coal on Louisiana parish bridges were calculated based on flexural and shear analyses performed in previous tasks. The span for most of these bridges was 20 ft. and the controlling factor was the high ratio of flexural moments. The results are presented in Appendix D. The estimated fatigue cost per trip is $1.05 for simply supported bridges.

CONCLUSIONS

1. The element of current weight laws that needs to be changed is the provision allowing tandem axle loads to approach 48,000 lb. This potential cost of this provision may approach $40 million annually. Currently, no permit fee addresses this provision. Bridge costs are the major contributor to this $40 million annual cost.
2. Lignite coal trucks, FHWA class 10 vehicles (3-S3), carrying 88,000 lb. GVW cause less damage to pavements than FHWA class 9 vehicles (3-S2) carrying 80,000 lb. GVW. The triple axle on the trailer reduces the impact of the extra 8,000 lb. as compared to the loads on the tandem axles of the 3-S2 vehicle.
3. Carrying 86,600 lb. GVW on a FHWA type 9 vehicle hauling timber causes more damage than is paid for by the $10/truck/year permit fee. Data developed in this study indicate that these vehicles induce pavement costs that amount to at least $346/year/vehicle, for equally loaded axles, assuming that all 10,626 of the harvest permit fees purchased in 2003 were purchased by log truck operators.
4. With the provision that an individual axle can carry 48-kips, the annual damage cost for timber trucks at a GVW of 86,600 lb., amounts to $4,324/yr/truck. Pavement costs increase from $346/yr/truck to $764/yr/truck, an increase of $418/yr/truck. Bridge costs with the 48-kip loads account for $3,560/yr/truck.
5. Raising the GVW to 100,000 lb. increases both pavement and bridge costs. Pavement costs increase to $704/yr/truck, for equally loaded axles, and bridge fatigue costs become very significant at $3,560/yr/truck, for a very low number of truck passages over bridges in a year. The total cost under scenario 3 amounts to $4,264/yr/truck.
6. Raising the GVW to 100,000 lb. while keeping the 48-kip axle load maximum increases the pavement costs over the equally loaded axle situation by $153/yr/truck. Bridge costs remain the same. The total cost under scenario 3 with the 48-kip axle provision amounts to $4,417/yr/truck.
7. Bridge fatigue costs for bridges on the state system, at both the 80,000 lb. GVW and 86,600 lb. GVW, are minimal, and the stresses induced do not exceed those from the design load. As a result, GVWs in this range may be applied without significant damage to existing bridges designed for the HS20-44 loading. Load-rated bridges and bridges designed to lower standards are subject to significant damage from loads of this magnitude.
8. Off-system bridges on parish roads are subject to substantial damage from trucks loaded to 80,000 lb. GVW and require further evaluations.
9. Very little coke fuel is transported on the Louisiana highway system. Investigators found only two refineries that shipped coke on Louisiana highways. As a result, we recommend no action on permits for transportation of coke fuel.

RECOMMENDATIONS

The recommendations presented in this section are limited by the time frame in senate resolution 123 that required DOTD to investigate and report back to the senate in March 2005. Since this project began in July 2004, the results from the study are limited by assumptions that were necessary to generate the information needed for the study. Among those limitations and assumptions were:

1. The accuracy of the pavement cross section data was generally limited to what district personnel knew about each control section. There was not sufficient time to consult plans when data was not available in the DOTD mainframe database.
2. The values of subgrade soil resilient modulus required in the overlay design procedure were most likely too large. A single average value of soil resilient modulus was used for each parish. This average value is too large for many control sections and would produce overlay thickness and costs that are too low.
3. The m-values used in the 1986 AASHTO pavement design guide were assumed to be 1.0. There was not sufficient time to attempt to develop values for each control section included in the study. The use of m = 1.0 produces overlay thicknesses and costs that are too low.
4. The traffic volumes included in the control section books are believed to be inaccurate. If these average daily traffic values are too small, the number of trucks predicted will be too small, and the design axle loads will be too small. The result is that the overlay thickness and costs will be too small.
5. Estimates of timber tonnage hauled on each of the 39 control sections included in the study were based on estimates of knowledgeable industry personnel and not from actual data taken from mill records. The accuracy of the data developed by the timber industry is consistent with the level of accuracy of much of the data on the pavement cross sections and ADT data.

Based on the limitations noted above, the following recommendations are based on the analysis performed during this study:

Based on the work accomplished in this project, the following recommendations are offered:

1. Eliminate the provision which permits individual axle loads to approach 48,000 lb (48-kips). Removing this provision can eliminate the accrual of $40 million/year in bridge damage.
2. The load limit on lignite coal should remain at 88,000 lb. GVW for the FHWA class 10 vehicle (3-S3).
3. The load limit on timber should remain at 86,600 lb. GVW for an FHWA class 9 vehicle (3-S2) but the 48-kip maximum axle load provision should be eliminated. However, the annual permit fee for equally loaded axles should be increased to at least $346/truck /year if the legislature desires equity for this type of vehicle. Additionally, the permit fee should be recalculated after the DOTD determines how

many of the 10,626 harvest permits issued in 2003 were purchased by log truck operators. The $346/truck/year fee was developed assuming all 10,626 permits were purchased by log truck operators.

4. If a timber truck operator modifies the axle configuration on the trailer from a tandem axle (FHWA class 9 vehicle, 3-S2) to a triple axle on the trailer (FHWA class 10 vehicle, 3-S3), the permit fee should remain at $10 and the load limit could be increased to 88,000 GVW for equal loads applied to the tractor tandem axle and the trailer triple axle.
5. Under no circumstances should the legislature consider increasing the GVW on FHWA class 9 vehicles (3-S2) to 100,000 lb. Annual pavement costs increase by $358/yr/truck when the GVW increases from 86,600 lb. to 100,000 lb. on the FHWA class 9 vehicle. However, the bridge costs at 100,000 lb. GVW become quite large and are estimated to be at least $3,560/yr/truck.
6. Off-system bridges are generally designed for lower loads than on-system bridges. As a result, the impact of trucks loaded to 80,000 lb. GVW can be very detrimental to the fatigue life of these bridges, and requires further evaluations.

ACRONYMS, ABBREVIATIONS, & SYMBOLS

18-k = 18,000 lb. axle load

3-S2 = truck with 3 axles on tractor and a semi-trailer with 2 axles

3-S3 = truck with 3 axles on tractor and a semi-trailer with 3 axles

A = annual cost, $

AASHTO = American Association of State Highway and Transportation Officials

ADT = average daily traffic, vehicles/day

a = a-value of a pavement material, the relative strength coefficient

B = the length of the axle group, in feet, used in bridge design

BC = binder course

D = thickness of a pavement layer, inches

DOTD = Louisiana Department of Transportation and Development

ESALs= equivalent 18-kip single axle loads

FHWA = Federal Highway Administration

F_{RL} = remaining life factor

ft = foot

GVW = gross vehicle weight

H_{OL} = Overlay thickness, inches

kip = 1,000 lb.

lb. = pound

LRFD = Load Resistance Factor Design

LTRC = Louisiana Transportation Research Center

M = mean or average of all observations in a data set

N = number of axles in a group

NPW = net present worth, $

n = size of a sample

O.C. = overlay cost, $

P_i = initial present serviceability index

P_t = terminal present serviceability index

P = present worth, $

PW = net present worth, $

psf = pounds per square foot

R = reliability level, %

S_o = overall standard deviation for construction of pavements

SN = structural number

SN_{OL} = structural number of an overlay

SN_{xeff} = total effective SN of the existing pavement above the subgrade

$SN_{xeff\text{-}rp}$ = effective structural capacity of all remaining pavement layers above the subgrade except for the existing PCC layer

TTI = Texas Transportation Institute

W = overall gross vehicle weight, lb.

WC = wearing course

$Z_{alpha/2}$ = value of standard normal deviate at an error level of alpha/2

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APPENDIX A
Bridges Considered for This Study

Table 1
Critical Bridges and Categories Considered in This Study

Critical Bridges for This Study		
	State Bridges	**Parish Bridges**
Category	**Number of Bridges**	**Number of Bridges**
Simple Beam	998	166
Continuous	149	1
Culvert	435	59
Others	75	20
Posted Bridges	169	302
Design Load Low (5, 10 ton)	55	3
Design Load Unknown	NA	394
Total	1881	945

Table 2
State Bridges Considered for Analysis

Trucks Hauling	**Category**	**Design Load HS20-44**		**Design Load H15**		**Total**
Timber		**Analyzed**	**Outliers**	**Analyzed**	**Outliers**	
	Simple Beam	787	13	191		991
	Continuous	126	19		2	147
Lignite Coal						
	Simple Beam	7				7
	Continuous	2				2
Total		922	32	191	2	1147

Table 3
Parish Bridges Considered for Analysis

Truck Hauling	**Category**	**Design Load HS20-44**	**Total**
Timber	Beam Simple	166	166
	Continuous	1	1
Total		167	167

APPENDIX B
Bridge Girder Evaluation Results

Table 1
Load Conditions for Simply Supported Bridge Girders

HS20-44 Truck Configuration		3S2 Truck Configuration	
Girder Span (ft.)	**Load on Girder**	**Girder Span (ft.)**	**Load on Girder**
20 To 28	P2 (or P3)	20 To 24	P1
20 To 28	P1 & P2	20 To 26	P2 & P3
20 To 28	P2 & P3	20 To 56	P4 & P5
33 To 120	P1, P2 & P3	24 To 62	P1, P2 & P3
33 To 120	P1, P2 & P3	50 To 57	P1, P2, P3 & P4
		52 To 120	P1, P2, P3, P4 & P5

Table 2
Absolute Maximum Moment, Shear and Deflection for H15 Truck Load

H15 (GVW 30 Kips)				
Span (ft.)	**Moment (K- ft.)**	**Shear (K)**	**Deflection*EI (ft.) (Due to Lane Load)**	**Deflection*EI (ft.) (Due to Truck Load)**
20	120.0	25.8	-4000.00	-4000.00
21	126.0	26.0	-4630.50	-4630.50
22	132.0	26.2	-5324.00	-5324.00
23	138.0	26.3	-6083.50	-6083.50
24	144.0	26.5	-6912.00	-6912.00
25	150.0	26.6	-7812.50	-7812.50
26	156.0	26.8	-8681.94	-8681.94
27	162.7	26.9	-9865.61	-9865.61
28	170.1	27.0	-11150.24	-11150.24
29	177.5	27.1	-12539.62	-12539.62
30	185.0	27.2	-14037.52	-14037.52
31	192.4	27.3	-15647.71	-15647.71
32	199.8	27.4	-17373.96	-17373.96
33	207.3	27.5	-19220.04	-19220.04
34	214.7	27.7	-21189.72	-21189.72
35	222.2	27.9	-23286.75	-23286.75
36	229.6	28.1	-25514.90	-25514.90
37	237.1	28.4	-27877.93	-27877.93
38	244.5	28.6	-30379.60	-30379.60
39	252.0	28.9	-33023.67	-33023.67
40	259.5	29.1	-35813.89	-35813.89
42	274.4	29.6	-41847.83	-41847.83

Table 2. Cont.
Absolute Maximum Moment, Shear and Deflection for H15 Truck Load

H15 (GVW 30 Kips)				
Span	**Moment**	**Shear**	**Deflection*EI (ft.)**	**Deflection*EI (ft.)**
(ft.)	**(K- ft.)**	**(K)**	**(Due to Lane Load)**	**(Due to Truck Load)**
44	289.3	30.1	-48511.46	-48511.46
46	304.3	30.5	-55834.84	-55834.84
48	319.2	31.0	-63847.98	-63847.98
50	334.2	31.5	-72580.91	-72580.91
52	349.1	32.0	-82063.67	-82063.67
54	364.1	32.5	-92326.27	-92326.27
56	379.1	32.9	-103398.72	-103398.72
58	397.6	33.4	-70728.10	-115311.05
60	418.5	33.9	-81000.00	-128093.26
62	439.9	34.4	-92352.10	-141775.37
64	461.8	34.9	-104857.60	-156387.38
66	484.1	35.3	-118592.10	-171959.31
68	506.9	35.8	-133633.60	-188521.16
70	530.3	36.3	-150062.50	-206102.94
75	590.6	37.5	-197753.91	-254716.48
80	654.0	38.7	-256000.00	-310360.95
85	720.4	39.9	-326253.91	-373505.13
90	789.8	41.1	-410062.50	-444617.84
95	862.1	42.3	-509066.41	-524167.86
100	937.5	43.5	-625000.00	-612623.96
110	1097.3	45.9	-915062.50	-818129.51
120	1269.0	48.3	-1296000.00	-1064884.63

Table 3
Absolute Maximum Moment, Shear and Deflection for HS20-44 Truck

HS20-44 (GVW 72 Kips)			
Span (ft.)	**Moment (K- ft.)**	**Shear (K)**	**Deflection*EI (ft.)**
20	160.0	41.6	-5333.33
21	168.0	42.7	-6174.00
22	176.0	43.6	-7098.67
23	184.0	44.5	-8111.33
24	192.7	45.3	-9135.61
25	207.4	46.1	-10920.96
26	222.2	46.8	-12903.44
27	237.0	47.4	-15091.36
28	252.0	48.0	-17493.00
29	267.0	48.8	-20116.60
30	282.1	49.6	-22970.36
31	297.3	50.3	-26062.45
32	312.5	51.0	-29401.04
33	327.8	51.6	-32994.26
34	343.5	52.2	-36815.69
35	361.2	52.8	-41262.90
36	378.9	53.3	-46024.81
37	396.6	53.8	-51110.45
38	414.3	54.3	-56528.84
39	432.1	54.8	-62289.00
40	449.8	55.2	-68399.93
42	485.3	56.0	-81710.22
44	520.9	56.7	-96531.82
46	556.5	57.4	-112936.81
48	592.2	58.0	-130997.28
50	627.8	58.6	-150785.28
52	663.5	59.1	-172372.87
54	699.3	59.6	-195832.10
56	735.0	60.0	-221235.00
58	770.8	60.4	-248653.61
60	806.5	60.8	-278159.96
62	842.3	61.2	-309826.06
64	878.1	61.5	-343723.96
66	913.9	61.8	-379925.66
68	949.8	62.1	-418503.18
70	985.6	62.4	-459528.53

Table 3 Cont.
Absolute Maximum Moment, Shear and Deflection for HS20-44 Truck

HS20-44 (GVW 72 Kips)			
Span (ft.)	**Moment (K- ft.)**	**Shear (K)**	**Deflection*EI (ft.)**
75	1075.2	63.0	-573273.80
80	1164.9	63.6	-703893.30
85	1254.6	64.1	-852512.17
90	1344.4	64.5	-1020255.53
95	1434.1	64.9	-1208248.44
100	1523.9	65.3	-1417615.97
110	1703.6	65.9	-1904975.13
120	1883.3	66.4	-2491333.31

Table 4
Absolute Maximum Moment, Shear and Deflection for 3S2 Truck

3S2 Truck			
Span (ft.)	**Moment (K-ft.)**	**Shear (K)**	**Deflection*EI (ft.)**
20	194.4	45.6	-7322.4
21	206.3	46.3	-8543.67
22	218.2	46.9	-9890.91
23	230.1	47.5	-11370.13
24	242.0	48.0	-12987.33
25	255.4	48.5	-14782.74
26	270.4	48.9	-16960.80
27	285.4	49.3	-19333.74
28	300.3	49.7	-21909.06
29	315.3	50.1	-24694.28
30	330.3	50.4	-27696.91
31	345.3	50.7	-30924.46
32	360.3	51.0	-34384.43
33	375.3	51.3	-38084.33
34	390.3	51.5	-42031.67
35	405.3	51.8	-46233.96
36	420.3	52.0	-50698.69
37	435.3	52.2	-55433.38
38	450.3	52.4	-60445.52
39	465.2	52.9	-65742.62
40	480.2	54.0	-71332.18
42	510.2	56.0	-83418.71
44	540.2	57.8	-96765.11
46	570.2	59.5	-111431.41
48	600.2	61.3	-127477.62
50	630.2	63.1	-144963.74
52	660.2	64.8	-163949.80
54	690.2	66.4	-184495.79
56	722.4	67.9	-208187.94
58	774.6	69.3	-242277.99
60	827.0	70.6	-279480.87
62	879.5	71.8	-319906.42
64	932.1	72.9	-363664.28
66	984.7	74.0	-410863.88
68	1037.5	75.0	-461614.46
70	1090.3	75.9	-516025.15
75	1222.6	78.1	-668779.84

Table 4 Cont.
Absolute Maximum Moment, Shear and Deflection for 3S2 Truck

3S2 Truck			
Span (ft.)	**Moment (K- ft.)**	**Shear (K)**	**Deflection*EI (ft.)**
80	1355.3	80.0	-846791.43
85	1488.2	81.6	-1051757.25
90	1621.3	83.1	-1285372.35
95	1754.7	84.4	-1549330.35
100	1888.2	85.6	-1845323.54
110	2155.6	87.6	-2540180.37
120	2423.5	89.3	-3383466.43

Table 5
Critical Location for Trucks on Continuous Bridge Girders

Span Length	**HS20-44**			**3S2**		
	Truck Location X (ft.) (From Left Support to Front Tire)			**Truck Location X (ft.) (From Left Support to Front Tire)**		
(ft.)	**Max Positive Moment**	**Max Negative Moment**	**Max Absolute Shear Force**	**Max Positive Moment**	**Max Negative Moment**	**Max Absolute Shear Force**
55	8	12	26	6	25	7
60	10	15	31	8	30	12
65	12	18	36	10	34	17
70	14	21	41	11	39	22
75	17	24	46	13	66 (a)	27
80	19	27	51	15	69 (a)	32
85	21	30	56	17	72 (a)	37
90	23	32	61	20	75 (a)	42
95	25	35	66	22	78 (a)	47
100	27	38	71	24	81 (a)	52
105	29	41	76	26	35	57
110	31	44	81	28	87 (a)	62
115	33	47	86	30	90 (a)	67
120	36	50	91	32	93 (a)	72
125	38	53	96	34	96 (a)	77
130	40	56	101	36	99(a)	82

(a) The Truck is moving from left to right along the bridge. Otherwise from right to left.

Table 6
Maximum Moments in Continuous Bridge Girder

Span Length (ft.)	HS20-44 Truck		3 S2 Truck	
	Max Positive Moment (Kip* ft.)	Max Negative Moment (Kip* ft.)	Max Positive Moment (Kip* ft.)	Max Negative Moment (Kip* ft.)
20	128.17	-114.82	163.43	-112.18
30	221.56	-167.49	262.61	-246.92
40	352.42	-239.39	381.88	-376.82
45	422.21	-274.03	442.17	-426.20
50	492.82	-316.15	502.70	-468.51
55	564.00	-357.46	564.39	-504.61
60	635.61	-398.13	650.70	-535.69
65	707.63	-438.28	757.74	-563.11
70	779.70	-478.02	857.95	-587.29
75	852.10	-517.41	959.68	-639.02
80	924.79	-556.51	1062.59	-705.45
85	997.59	-595.38	1166.47	-770.74
90	1070.49	-634.07	1271.17	-835.06
95	1143.45	-672.60	1376.59	-898.52
100	1216.48	-710.98	1482.56	-961.26
105	1289.55	-749.22	1588.99	-1052.50
110	1362.67	-787.36	1695.83	-1084.89
115	1435.83	-825.38	1802.99	-1145.90
120	1509.14	-863.32	1910.45	-1206.54
125	1582.49	-901.18	2018.16	-1266.75
130	1655.86	-938.97	2126.08	-1326.61

Table 7
Maximum Absolute Shear Forces in Continuous Bridge Girder

Span Length (ft.)	HS20-44 Truck Max Absolute Shear (Kip)	3S2 Truck Max Absolute Shear (Kip)
20	41.65	44.59
30	51.31	52.05
40	57.60	56.62
45	59.66	61.78
50	61.24	66.29
55	62.48	71.03
60	63.49	75.05
65	64.31	78.36
70	65.00	81.13
75	65.58	83.47
80	66.07	85.47
85	66.50	87.20
90	66.87	88.70
95	67.20	90.01
100	67.49	91.17
105	67.75	92.20
110	67.98	93.12
115	68.19	93.95
120	68.38	94.69
125	68.55	95.36
130	68.71	95.98

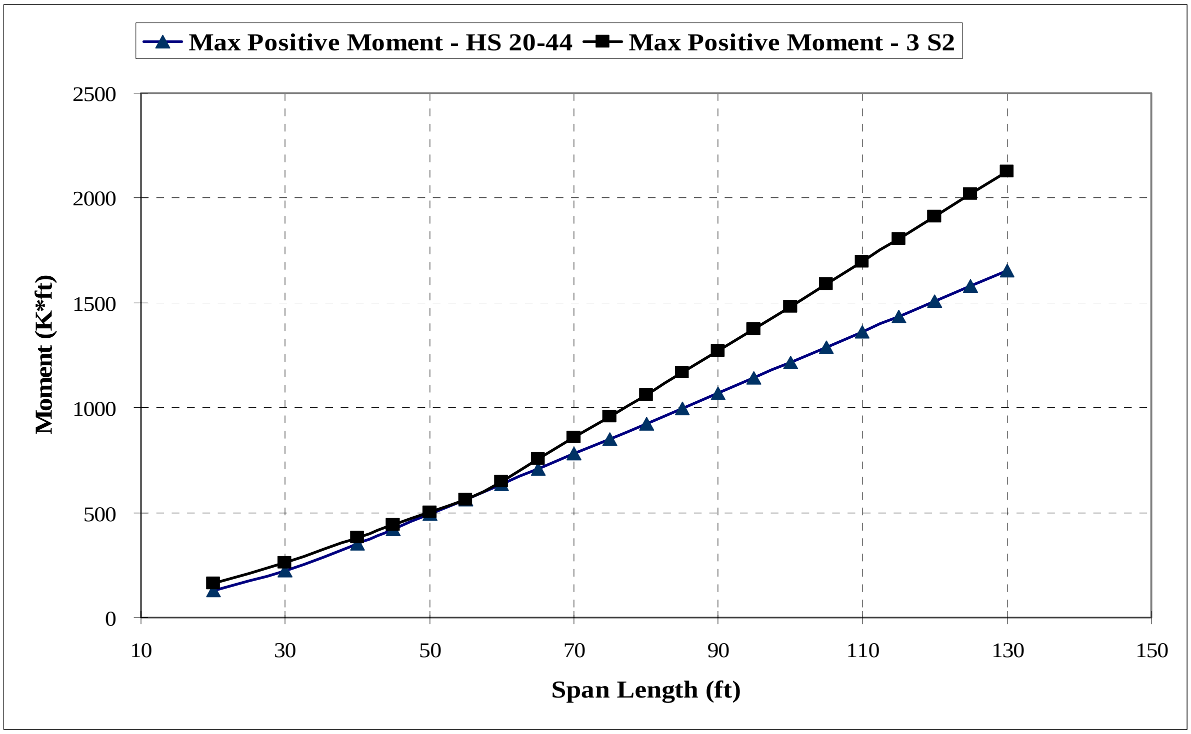

Figure 1
Maximum Positive Moment in Continuous Bridge Girders

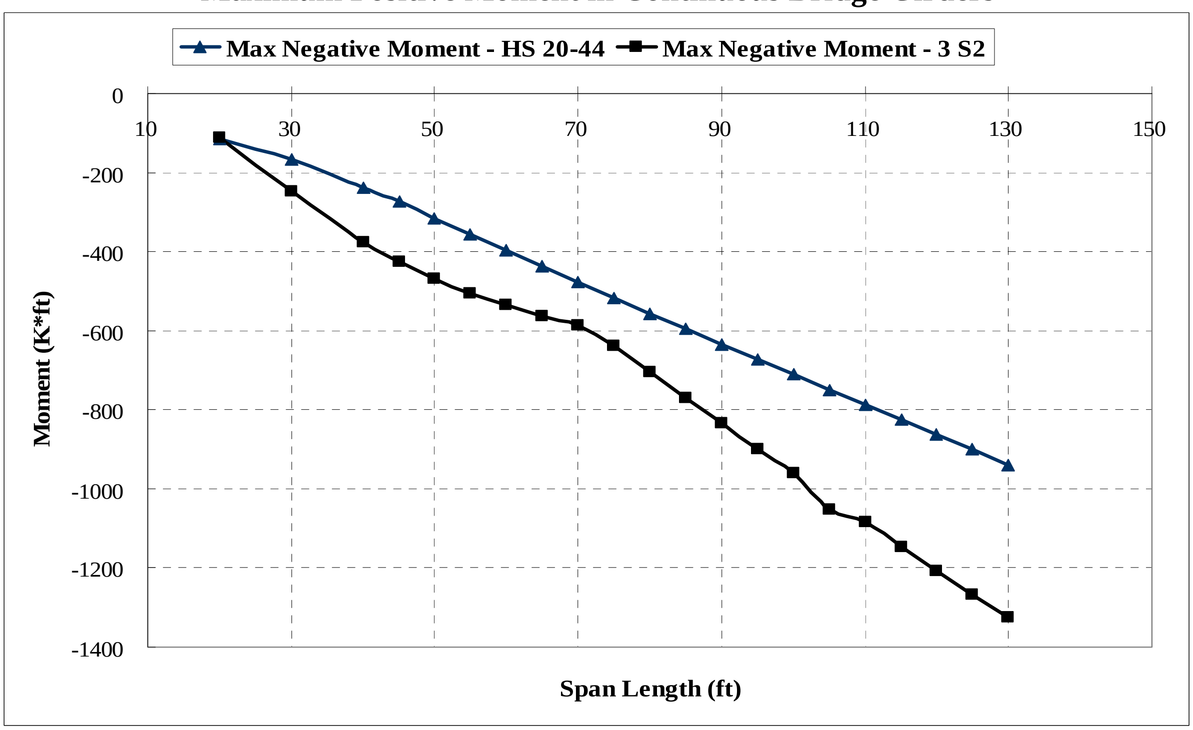

Figure 2
Maximum Negative Moment in Continuous Bridge Girders

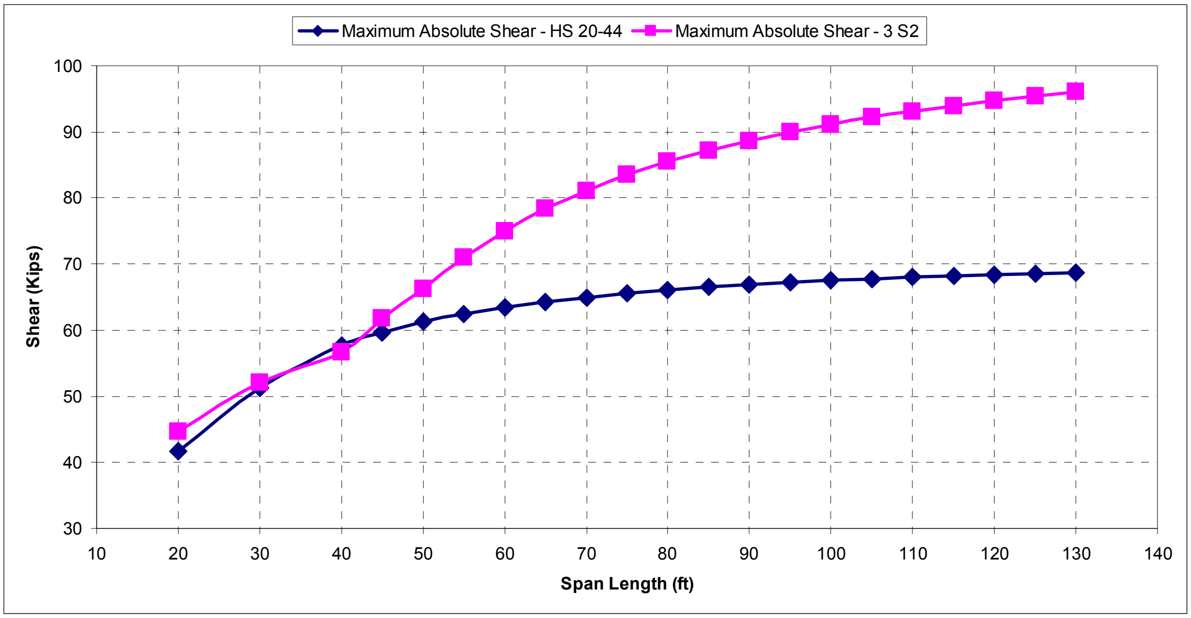

Figure 3
Maximum Absolute Shear Forces in Continuous Bridges

Table 8
Governing Load Conditions For H15 Truck

Span Length (ft.)	Governing Load Conditions For H15 Truck		
	For Moment	**For Shear**	**For Deflection**
20 to 32	Standard Truck Load	Standard Truck Load	Standard Truck Load
33 to 56	Standard Truck Load	Standard Lane Load	Standard Truck Load
57 to 120	Standard Lane Load	Standard Lane Load	Standard Truck Load

Table 9
Effects of 3S2 Truck Loads on Simple Span Bridge Girders

Span (ft.)	3S2/H15			3S2/HS20-44		
	Moment	Shear	Deflection	Moment	Shear	Deflection
20	1.62	1.77	1.83	1.22	1.1	1.37
21	1.64	1.78	1.85	1.23	1.08	1.38
22	1.65	1.79	1.86	1.24	1.08	1.39
23	1.67	1.81	1.87	1.25	1.07	1.4
24	1.68	1.81	1.88	1.26	1.06	1.42
25	1.7	1.82	1.89	1.23	1.05	1.35
26	1.73	1.83	1.95	1.22	1.05	1.31
27	1.75	1.83	1.96	1.2	1.04	1.28
28	1.77	1.84	1.96	1.19	1.04	1.25
29	1.78	1.85	1.97	1.18	1.03	1.23
30	1.79	1.85	1.97	1.17	1.02	1.21
31	1.79	1.86	1.98	1.16	1.01	1.19
32	1.8	1.86	1.98	1.15	1	1.17
33	1.81	1.86 (a)	1.98	1.15	0.99	1.15
34	1.82	1.86 (a)	1.98	1.14	0.99	1.14
36	1.83	1.85 (a)	1.99	1.11	0.98	1.1
37	1.84	1.84 (a)	1.99	1.1	0.97	1.08
38	1.84	1.83 (a)	1.99	1.09	0.97	1.07
39	1.85	1.83 (a)	1.99	1.08	0.97	1.06
40	1.85	1.86 (a)	1.99	1.07	0.98	1.04
42	1.86	1.89 (a)	1.99	1.05	1	1.02
44	1.87	1.92 (a)	1.99	1.04	1.02	1
46	1.87	1.95 (a)	2	1.02	1.04	0.99
48	1.88	1.98 (a)	2	1.01	1.06	0.97
50	1.89	2.00 (a)	2	1	1.08	0.96
52	1.89	2.03 (a)	2	0.99	1.1	0.95
54	1.9	2.04 (a)	2	0.99	1.12	0.94
56	1.91	2.06 (a)	2.01	0.98	1.13	0.94
58	1.95 (a)	2.08 (a)	2.1	1.01	1.15	0.97
60	1.98 (a)	2.08 (a)	2.18	1.03	1.16	1
62	2.00 (a)	2.09 (a)	2.26	1.04	1.17	1.03
64	2.02 (a)	2.09 (a)	2.33	1.06	1.19	1.06
66	2.03 (a)	2.10 (a)	2.39	1.08	1.2	1.08
68	2.05 (a)	2.09 (a)	2.45	1.09	1.21	1.1
70	2.06 (a)	2.09 (a)	2.5	1.11	1.22	1.12
75	2.07 (a)	2.08 (a)	2.63	1.14	1.24	1.17

Table 9 Cont.
Effects of 3S2 Truck Loads on Simple Span Bridge Girders

Span (ft.)	3S2/H15			3S2/HS20-44		
	Moment	Shear	Deflection	Moment	Shear	Deflection
80	2.07 (a)	2.07 (a)	2.73	1.16	1.26	1.2
85	2.07 (a)	2.05 (a)	2.82	1.19	1.27	1.23
90	2.05 (a)	2.02 (a)	2.89	1.21	1.29	1.26
95	2.04 (a)	1.99 (a)	2.96	1.22	1.3	1.28
100	2.01 (a)	1.97 (a)	3.01	1.24	1.31	1.3
110	1.96 (a)	1.91 (a)	3.1	1.27	1.33	1.33
120	1.91 (a)	1.85 (a)	3.18	1.29	1.34	1.36

(a) Maximum value determined by lane load.

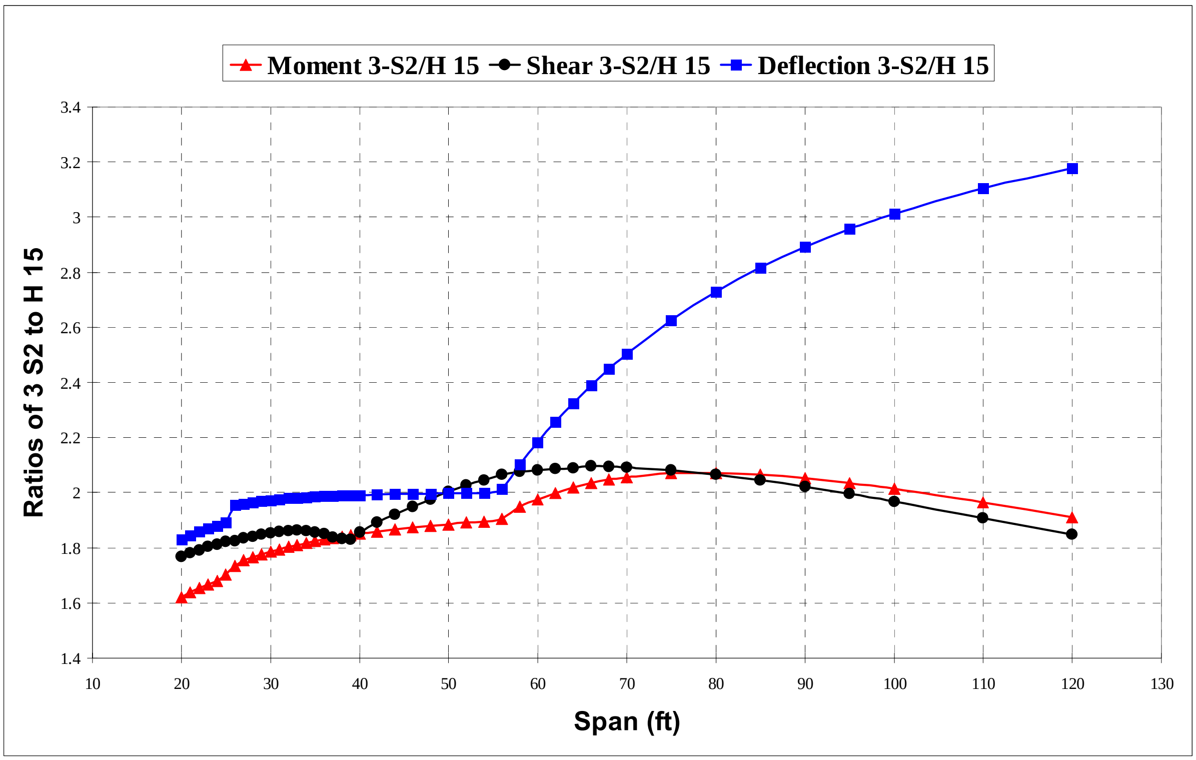

Figure 4
Effects of 3S2 Truck on Simple Span Bridges with H15 Design Loads

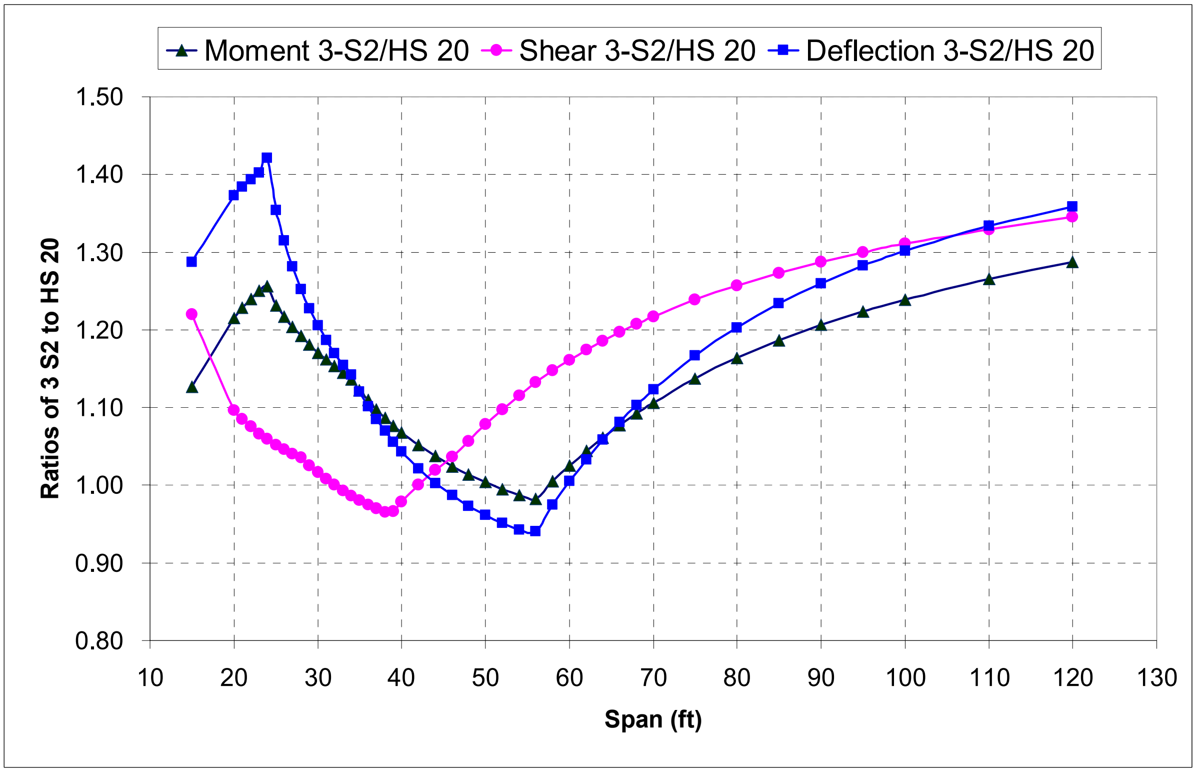

Figure 5
Effects of 3S2 Truck on Simple Span Bridges with HS20-44 Design Loads

Table 10
Ratio of Max. Positive and Negative Moment for 3S2 and HS20-44 Truck

Span Length	Ratio 3S2/HS20-44		
(ft.)	Positive Moment	Negative Moment	Shear
20	1.28	0.98	1.07
30	1.19	1.47	1.01
40	1.08	1.57	0.98
45	1.05	1.56	1.04
50	1.02	1.48	1.08
55	1	1.41	1.14
60	1.02	1.35	1.18
65	1.07	1.28	1.22
70	1.1	1.23	1.25
75	1.13	1.24	1.27
80	1.15	1.27	1.29
85	1.17	1.29	1.31
90	1.19	1.32	1.33
95	1.2	1.34	1.34
100	1.22	1.35	1.35
105	1.23	1.4	1.36
110	1.24	1.38	1.37
115	1.26	1.39	1.38
120	1.27	1.4	1.38
125	1.28	1.41	1.39
130	1.28	1.41	1.40

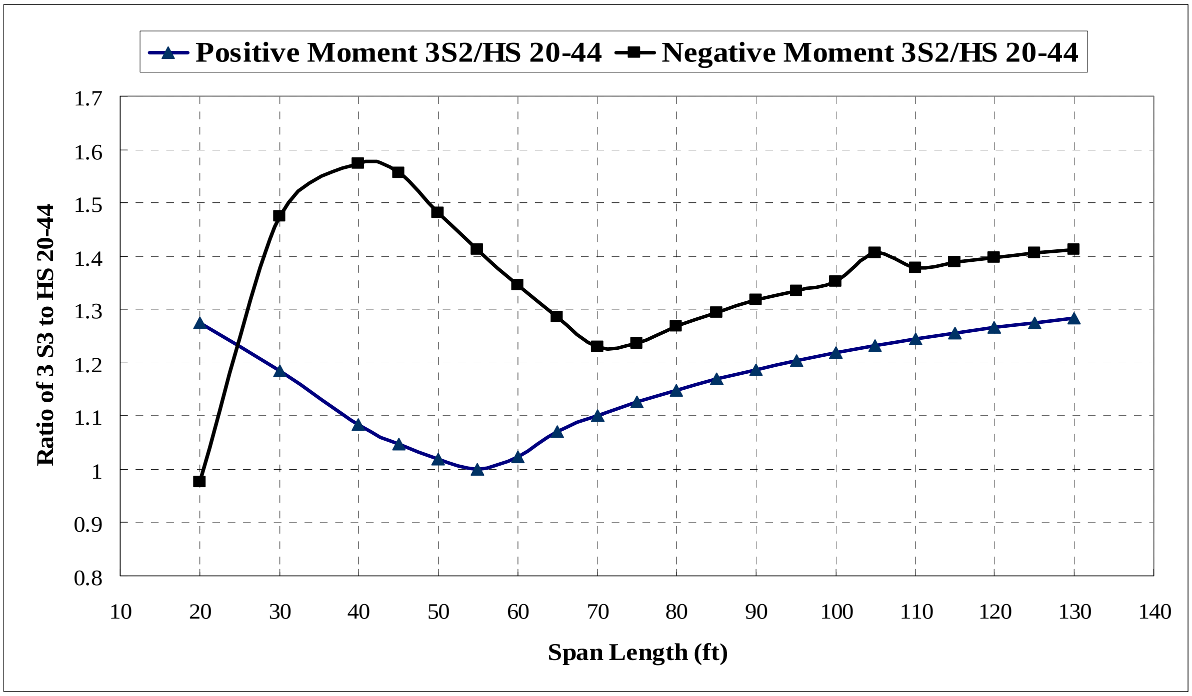

Figure 6
Effects on Moments of 3S2 Truck on Continuous Bridges

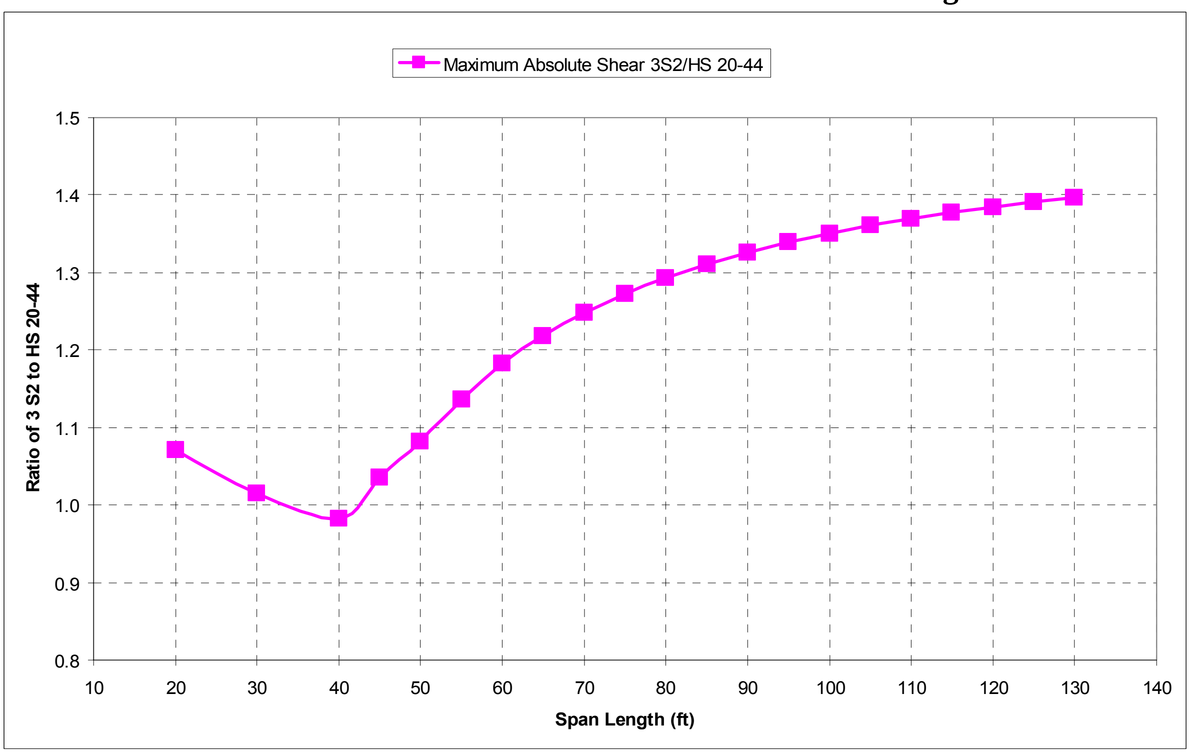

Figure 7
Effects on Shear Forces of 3S2 Truck on Continuous Bridges

Table 11
State Bridges Simply Supported with Design Load H15

Max Span Length (ft.)	Number of Bridges Design Load H15	Ratio 3S2/H 15		
		Moment	Shear	Deflection
20 or shorter	160	1.62	1.77	1.83
25	17	1.70	1.82	1.89
30	5	1.79	1.85	1.97
35	1	1.82	1.86	1.99
40	1	1.85	1.86	1.99
46	1	1.87	1.95	2.00
50	1	1.89	2.00	2.00
56	1	1.91	2.06	2.01
75	2	2.07	2.08	2.63
80	2	2.07	2.07	2.73
	Total (191)			

Table 12

State Bridges Simply Supported with Design Load HS20-44

Max Span Length (ft.)	Number of Bridges Design Load HS20-44	Ratio 3 S2/HS20-44		
		Moment	Shear	Deflection
20 or shorter	632	1.22	1.1	1.37
25	30	1.23	1.05	1.35
30	1	1.17	1.02	1.21
35	7	1.12	0.98	1.12
40	14	1.07	0.98	1.04
46	15	1.02	1.04	0.99
50	16	1	1.08	0.96
56	3	0.98	1.13	0.94
60	12	1.03	1.16	1
66	4	1.08	1.2	1.08
70	17	1.11	1.22	1.12
75	7	1.14	1.24	1.17
80	2	1.16	1.26	1.2
85	5	1.19	1.27	1.23
90	5	1.21	1.29	1.26
95	4	1.22	1.3	1.28
100	6	1.24	1.31	1.3
110	5	1.27	1.33	1.33
120	2	1.29	1.34	1.36
125	4	Outliers		
130	1			
135	1			
140	2			
145	1			
170	2			
180	1			
235	1			
	Total (800)			

Table 13
State Bridges Continuous with Design Load HS20-44

Max Span Length (ft.)	Number of Bridges Design Load HS20-44	Ratio 3S2/HS20-44		
		Positive Moment	Negative Moment	Shear
20	3	1.28	0.98	1.07
40	1	1.08	1.57	0.98
45	1	1.05	1.56	1.04
50	14	1.02	1.48	1.08
55	1	1.00	1.41	1.14
60	4	1.02	1.35	1.18
65	6	1.07	1.28	1.22
70	15	1.10	1.23	1.25
75	10	1.13	1.24	1.27
80	2	1.15	1.27	1.29
85	5	1.17	1.29	1.31
90	18	1.19	1.32	1.33
95	3	1.20	1.34	1.34
100	13	1.22	1.35	1.35
105	20	1.23	1.40	1.36
110	2	1.24	1.38	1.37
120	2	1.27	1.40	1.38
125	4	1.28	1.41	1.39
130	2	1.28	1.41	1.40
135	3	Outliers		
140	1			
145	2			
150	2			
160	1			
165	1			
175	4			
180	1			
190	1			
200	0			
330	1			
335	1			
375	1			
	Total (145)			

APPENDIX C
Bridge Deck Evaluation Results

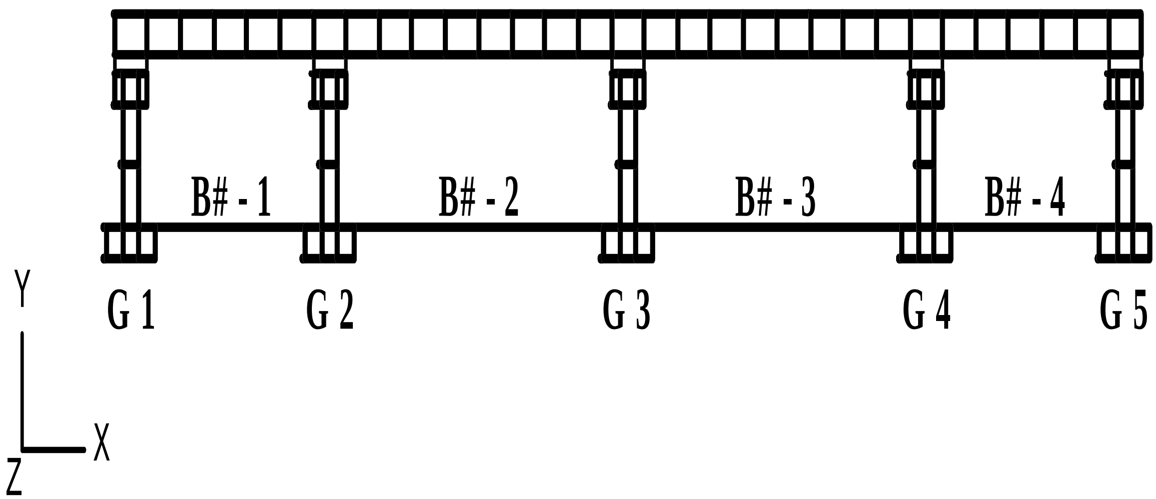

Figure 1
Models Used for Bridge Deck Analysis

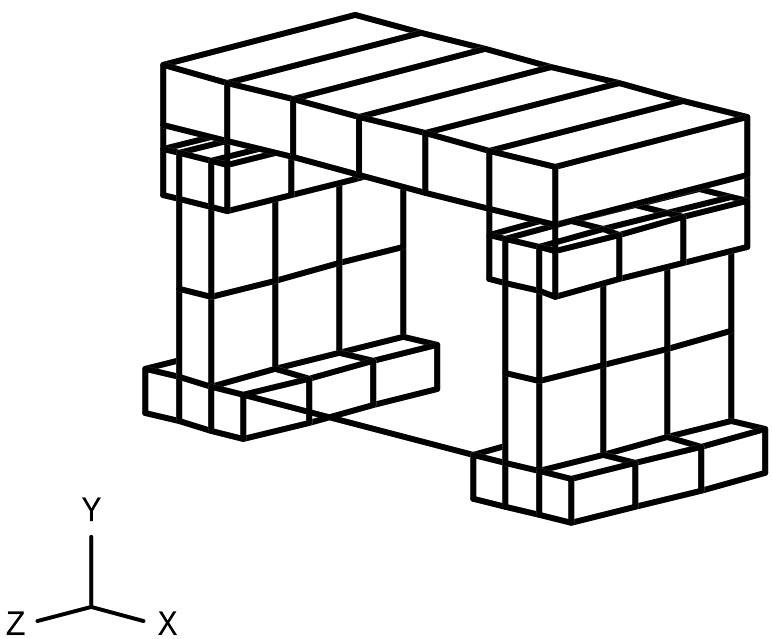

Figure 2
Typical Plate and Girder Elements

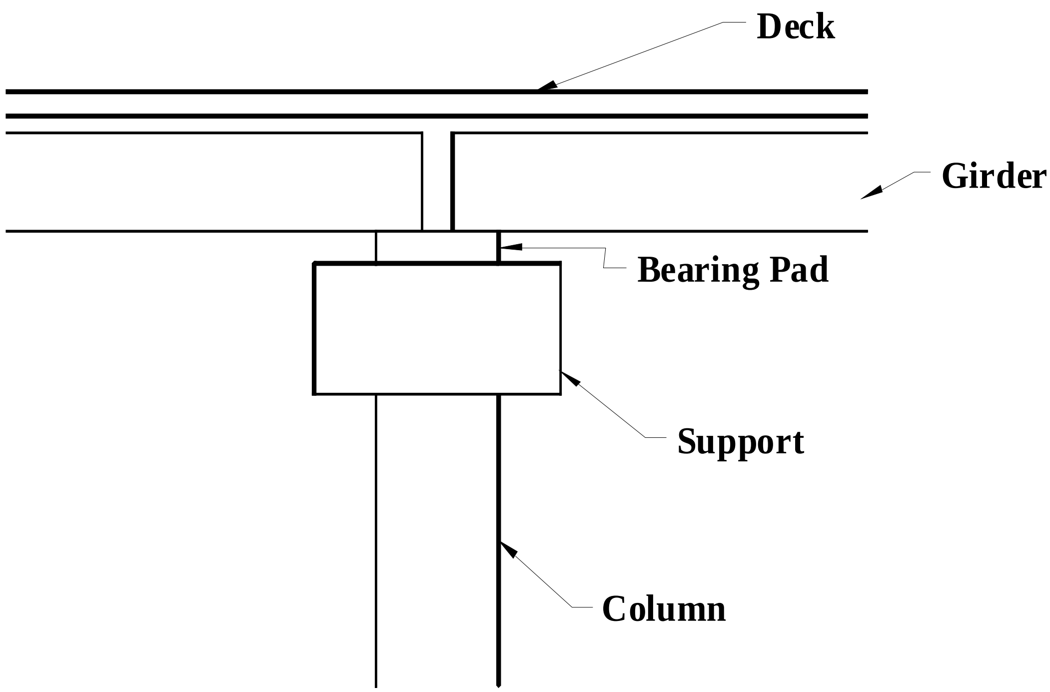

Figure 3
Elevation View of Girders over Interior Support

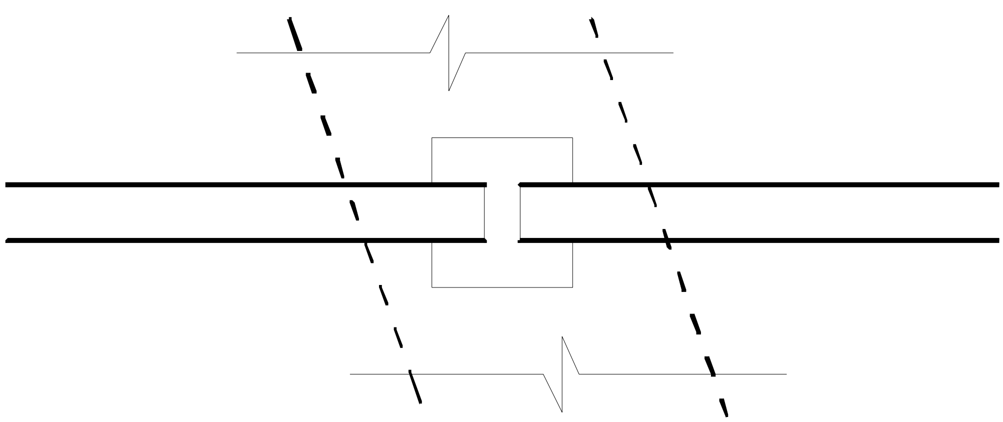

Figure 4
Plan View of Girders over Interior Support

Table 1
Top Surface of Continuous Bridge Deck for HS20-44 Truck Loads

HS20-44						
Span Length	Max Value of Stress (Ksi)					
(ft.)	Max Tensile Stress			Max Compressive Stress		
	Longitudinal	*Transverse*	*Shear*	*Longitudinal*	*Transverse*	*Shear*
20	0.060	0.216	0.038	-0.309	-0.329	-0.038
30	0.099	0.193	0.045	-0.353	-0.424	-0.057
45	0.127	0.114	0.076	-0.317	-0.432	-0.061
60	0.184	0.127	0.075	-0.337	-0.495	-0.065
75	0.254	0.146	0.068	-0.352	-0.534	-0.066
90	0.363	0.191	0.075	-0.363	-0.554	-0.086
105	0.476	0.231	0.088	-0.373	-0.564	-0.084
120	0.590	0.267	0.102	-0.383	-0.568	-0.108

Table 2
Top Surface of Continuous Bridge Deck for 3S2 Truck Loads

3S2						
Span Length	Max Value of Stress (Ksi)					
(ft.)	Max Tensile Stress			Max Compressive Stress		
	Longitudinal	*Transverse*	*Shear*	*Longitudinal*	*Transverse*	*Shear*
20	0.055	0.156	0.060	-0.222	-0.317	-0.085
30	0.114	0.137	0.057	-0.204	-0.380	-0.065
45	0.222	0.121	0.066	-0.223	-0.435	-0.060
60	0.295	0.148	0.073	-0.240	-0.471	-0.064
75	0.317	0.187	0.084	-0.263	-0.548	-0.099
90	0.493	0.253	0.101	-0.396	-0.589	-0.112
105	0.660	0.308	0.118	-0.304	-0.622	-0.118
120	0.844	0.366	0.140	-0.382	-0.621	-0.149

Table 3
Bottom Surface of Continuous Bridge Deck for HS20-44 Truck Loads

HS20-44						
Span Length	**Max Value of Stress (Ksi)**					
(ft.)	**Max Tensile Stress**			**Max Compressive Stress**		
	Longitudinal	***Transverse***	***Shear***	***Longitudinal***	***Transverse***	***Shear***
20	0.309	0.329	0.038	-0.060	-0.216	-0.038
30	0.353	0.424	0.057	-0.099	-0.193	-0.045
45	0.317	0.432	0.061	-0.127	-0.114	-0.076
60	0.337	0.495	0.065	-0.184	-0.127	-0.075
75	0.352	0.534	0.066	-0.254	-0.146	-0.068
90	0.363	0.554	0.086	-0.363	-0.191	-0.075
105	0.373	0.564	0.084	-0.476	-0.231	-0.088
120	0.383	0.568	0.108	-0.590	-0.267	-0.102

Table 4
Bottom Surface of Continuous Bridge Deck for 3S2 Truck Loads

3S2						
Span Length	**Max Value of Stress (Ksi)**					
(ft.)	**Max Tensile Stress**			**Max Compressive Stress**		
	Longitudinal	***Transverse***	***Shear***	***Longitudinal***	***Transverse***	***Shear***
20	0.222	0.317	0.085	-0.055	-0.156	-0.060
30	0.204	0.380	0.065	-0.114	-0.137	-0.057
45	0.223	0.435	0.060	-0.222	-0.121	-0.066
60	0.240	0.471	0.064	-0.295	-0.148	-0.073
75	0.263	0.548	0.099	-0.317	-0.187	-0.084
90	0.396	0.589	0.112	-0.493	-0.253	-0.101
105	0.304	0.622	0.118	-0.660	-0.308	-0.118
120	0.382	0.621	0.149	-0.844	-0.366	-0.140

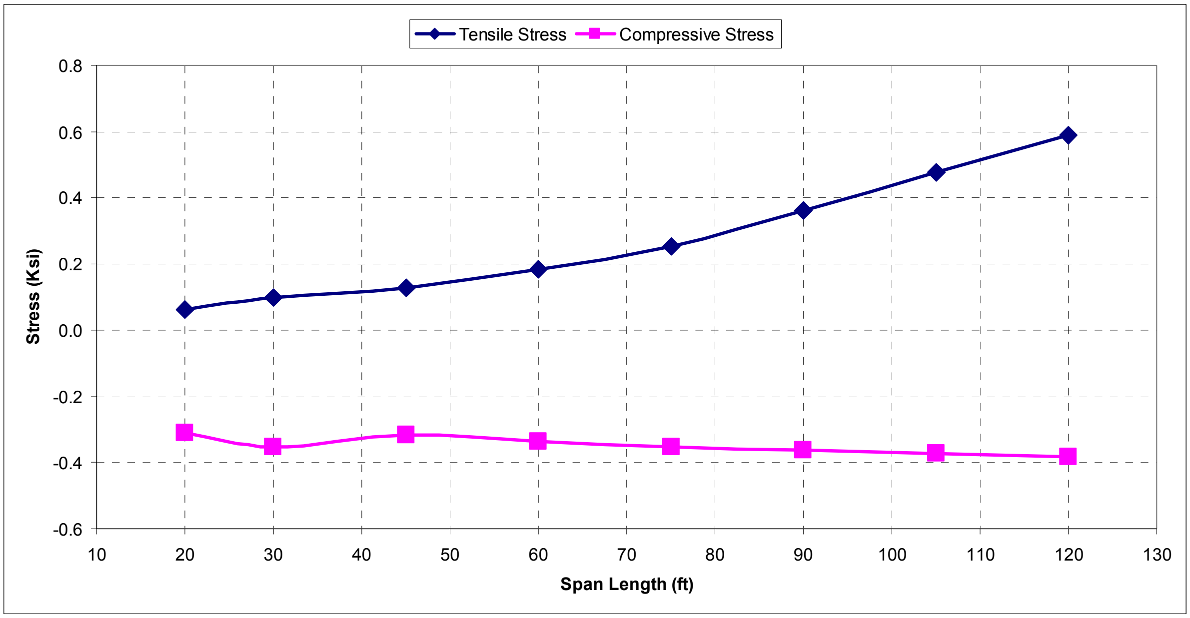

Figure 5
Maximum Longitudinal Stress of HS20-44 Truck at Top Surface of the Deck

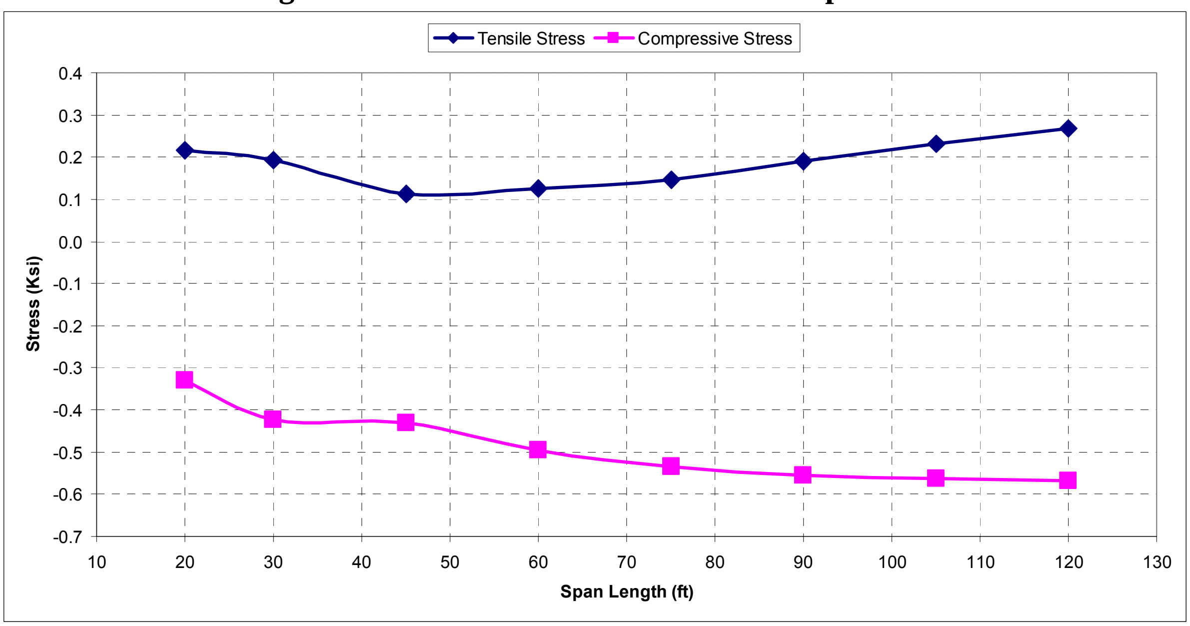

Figure 6
Maximum Transverse Stress of HS20-44 Truck at Top Surface of the Deck

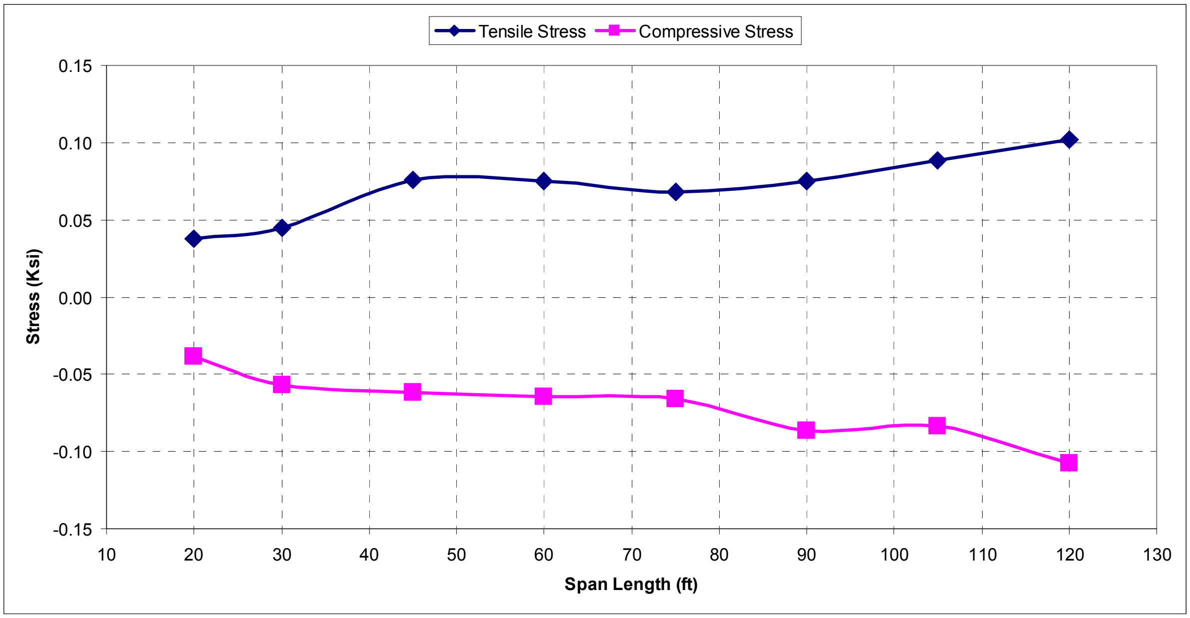

Figure 7
Maximum Shear Stress of HS20-44 Truck at Top Surface of the Deck

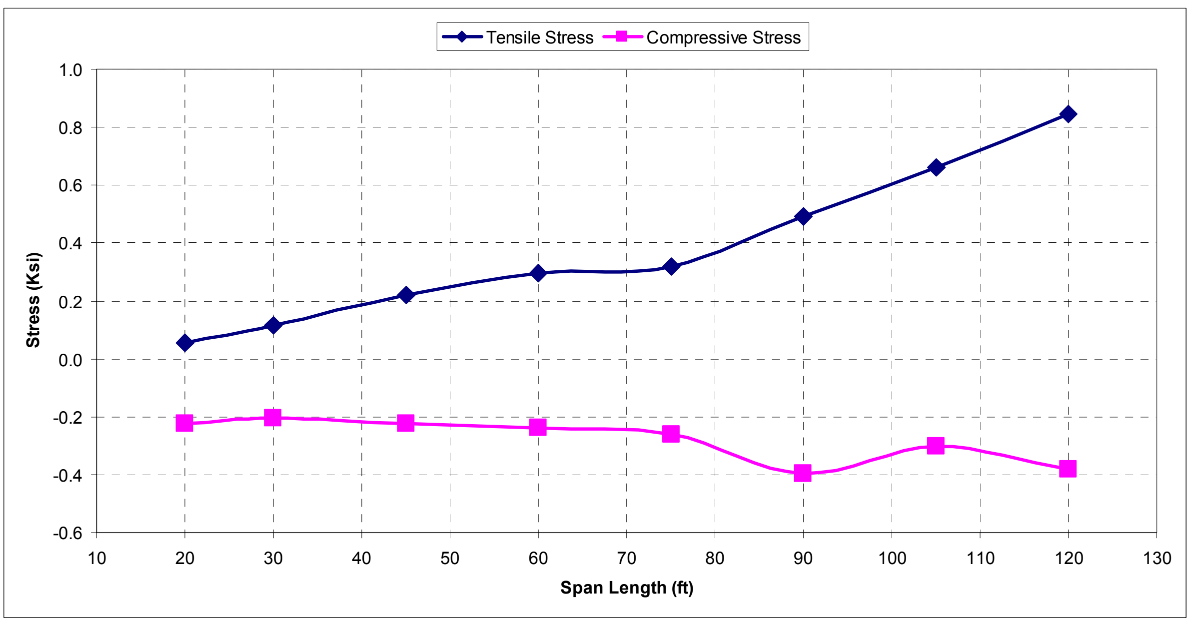

Figure 8
Maximum Longitudinal Stress of 3S2 Truck at Top Surface of the Deck

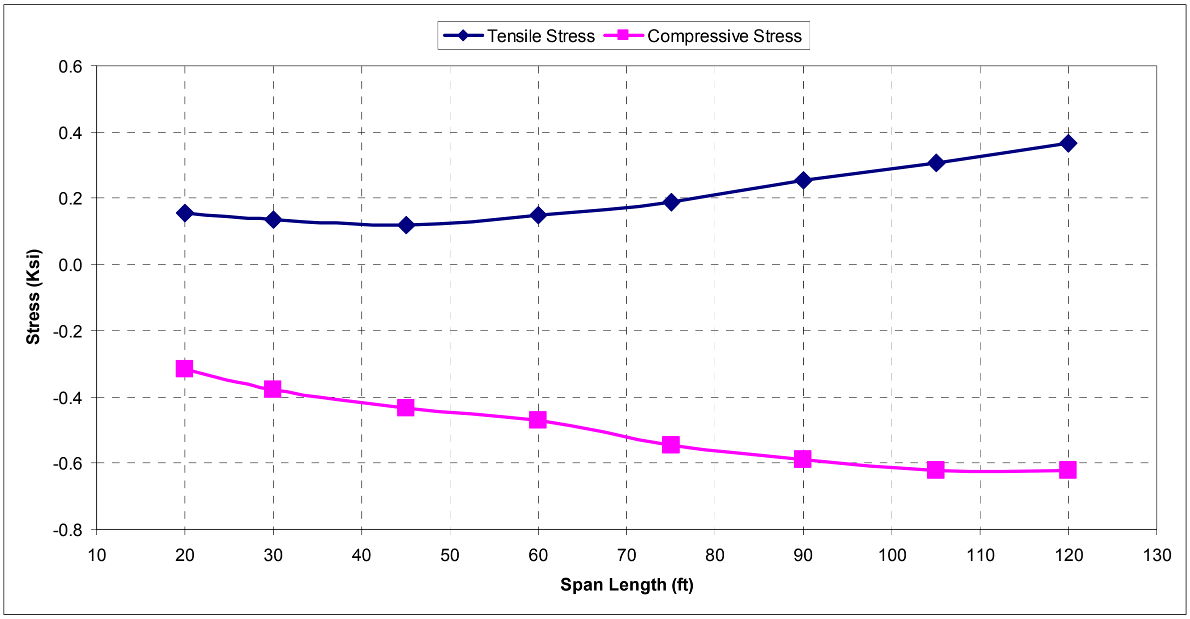

Figure 9
Maximum Transverse Stress of 3S2 Truck at Top Surface of the Deck

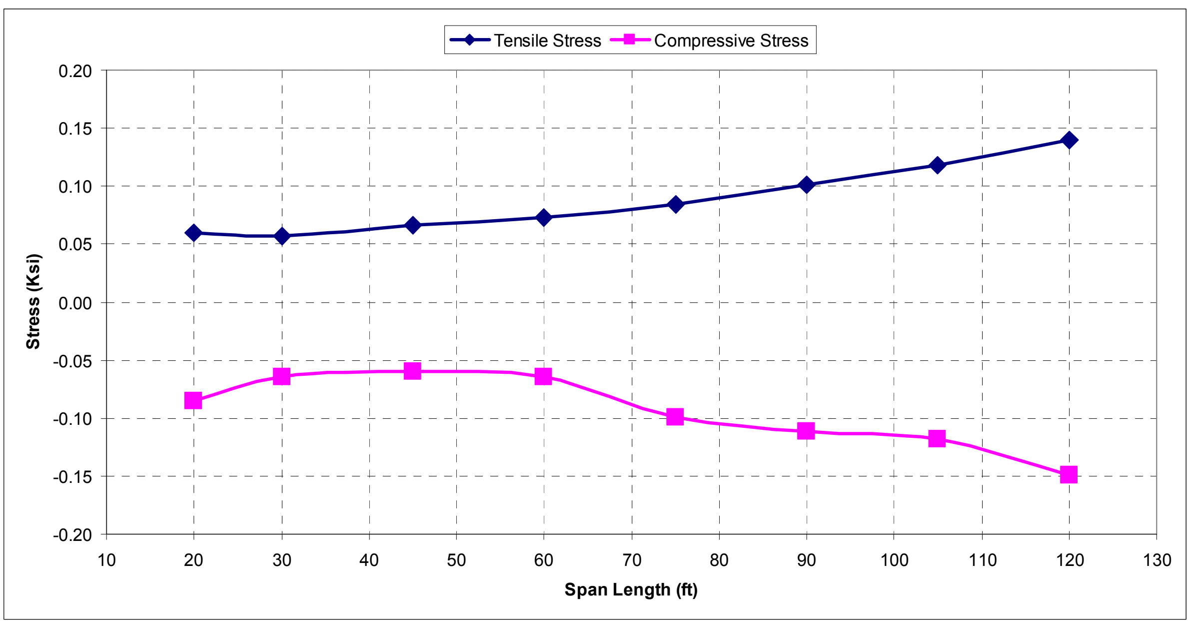

Figure 10
Maximum Shear Stress of 3S2 Truck at Top Surface of the Deck

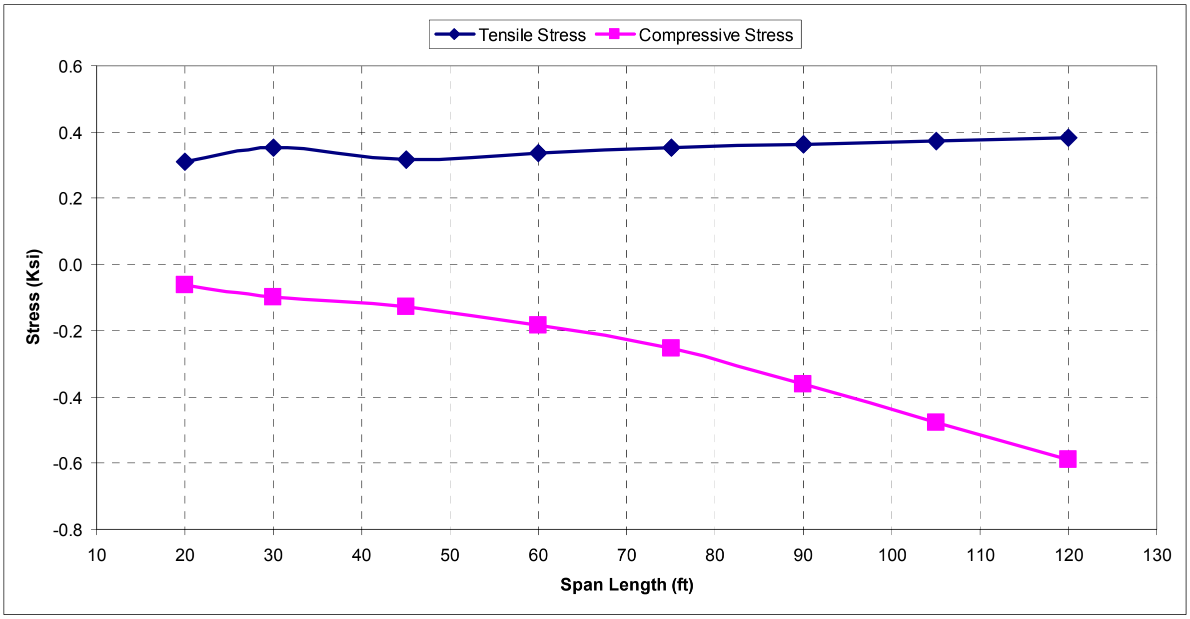

Figure 11
Maximum Longitudinal Stress of HS20-44 Truck at Bottom Surface of the Deck

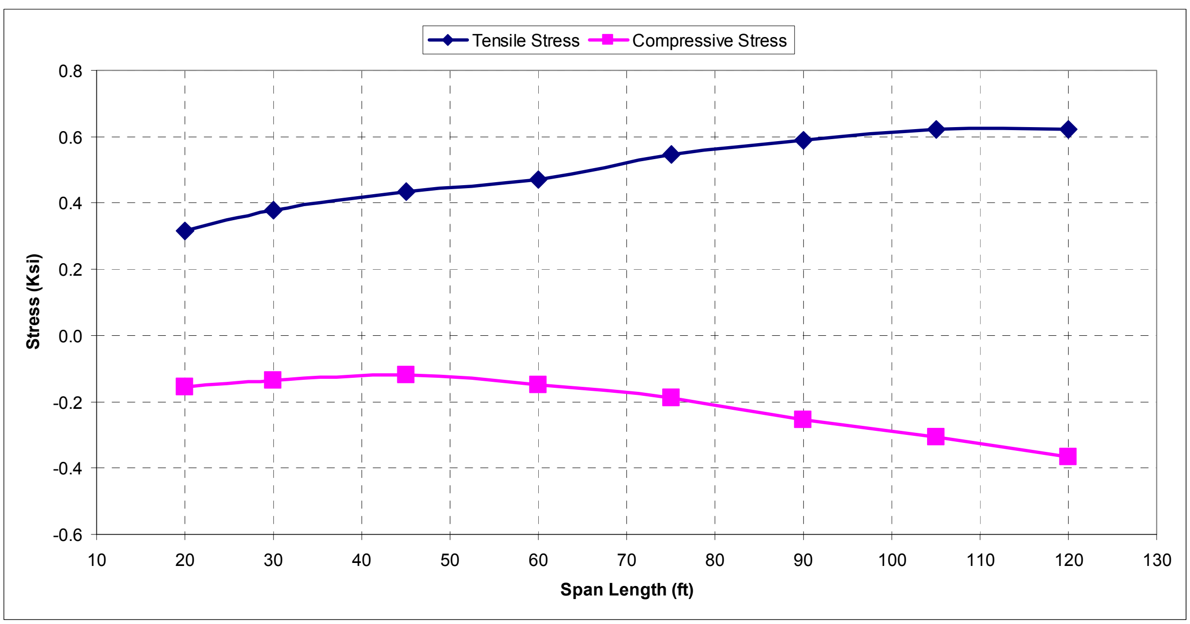

Figure 12
Maximum Transverse Stress of HS20-44 Truck at Bottom Surface of the Deck

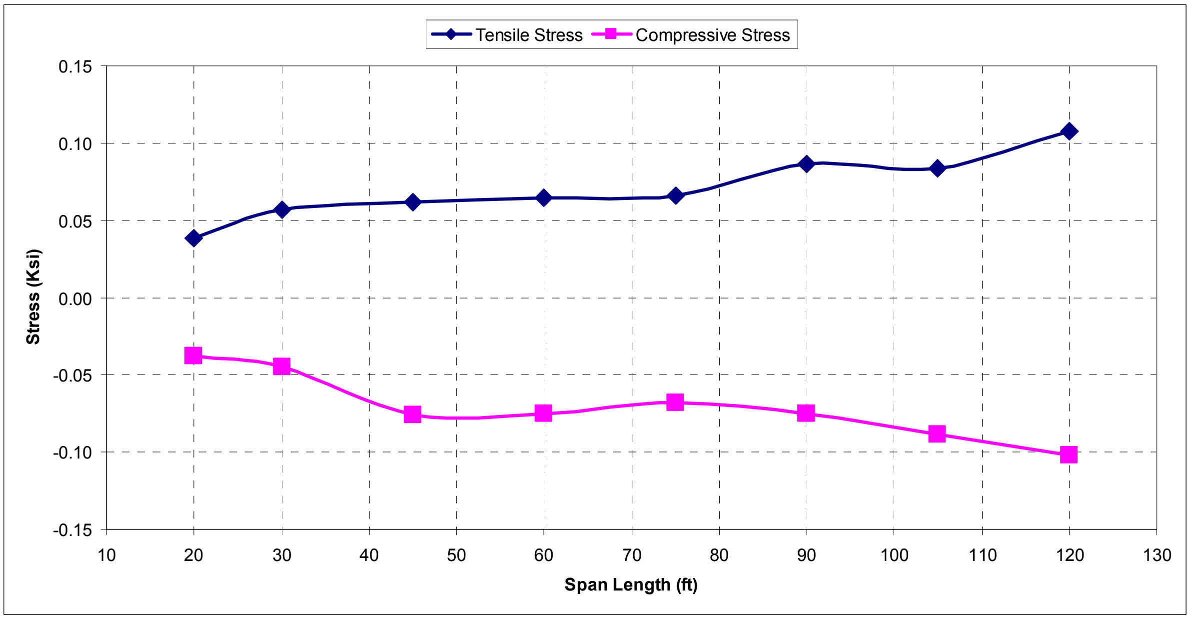

Figure 13
Maximum Shear Stress of HS20-44 Truck at Bottom Surface of the Deck

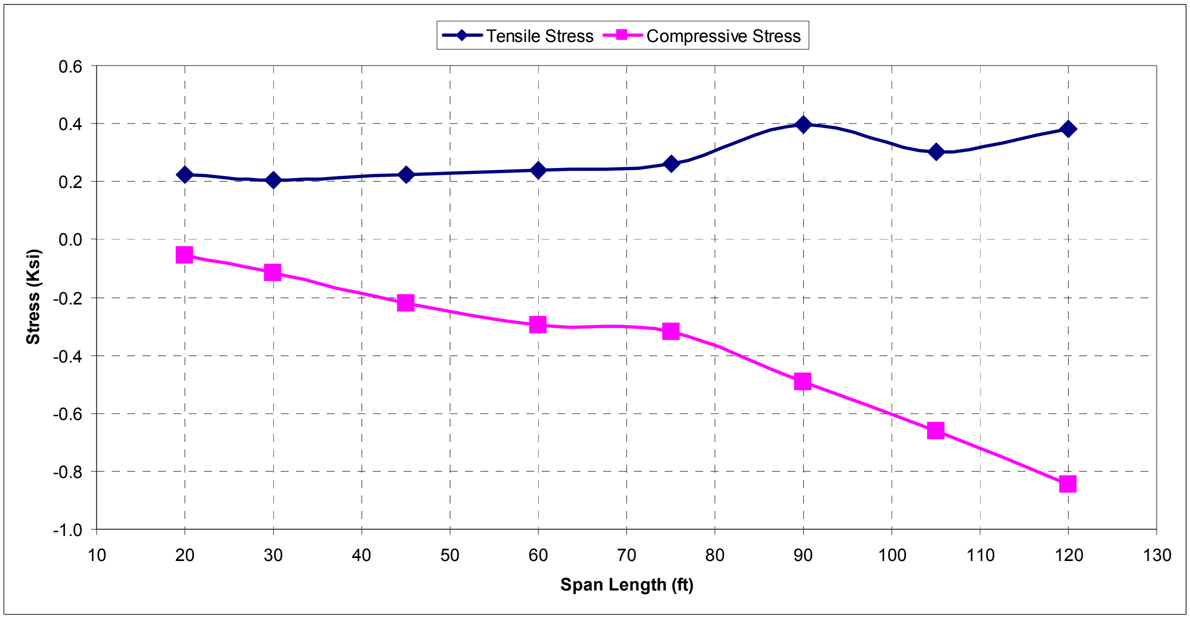

Figure 14
Maximum Longitudinal Stress of 3S2 Truck at Bottom Surface of the Deck

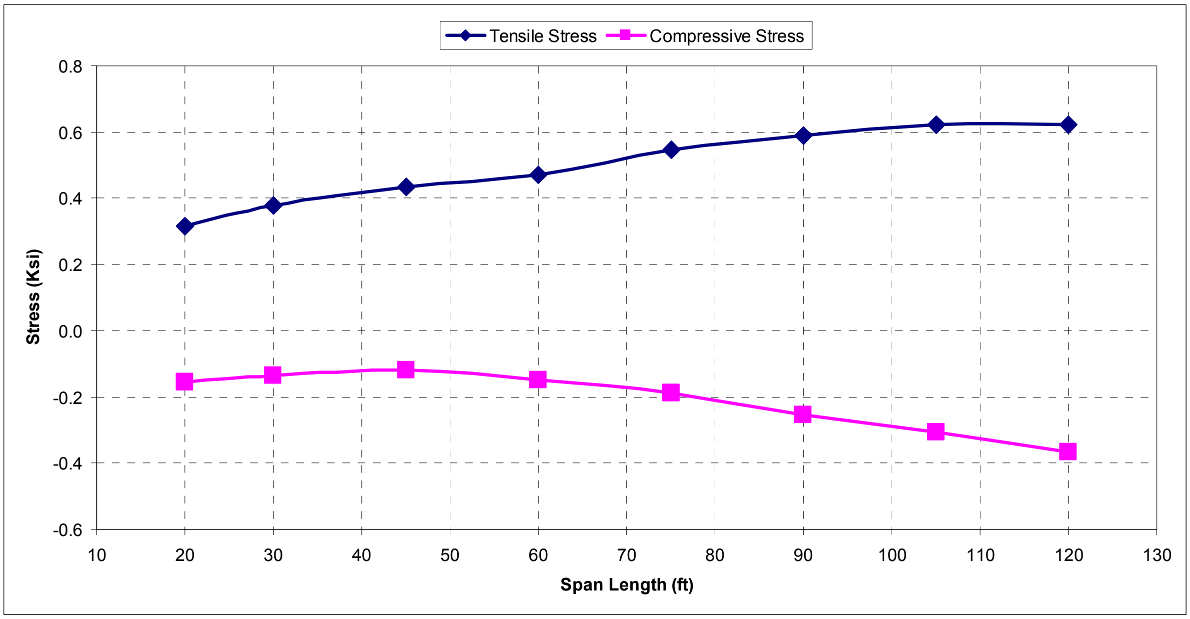

Figure 15
Maximum Transverse Stress of 3S2 Truck at Bottom Surface of the Deck

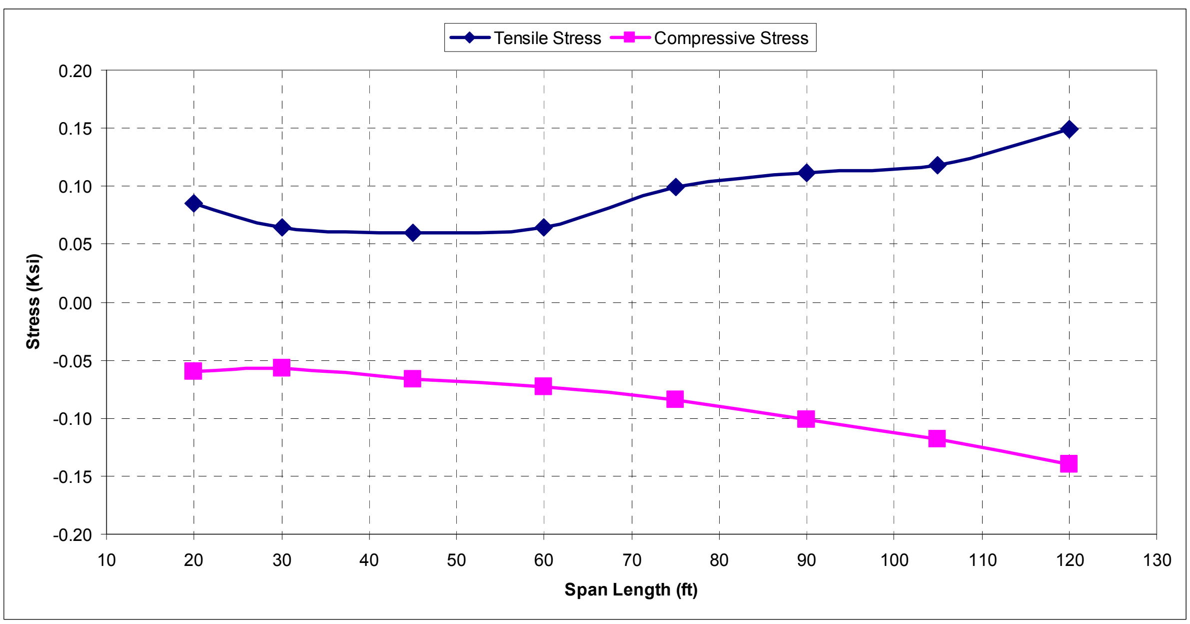

Figure 16
Maximum Shear Stress of 3S2 Truck at Bottom Surface of the Deck

Table 5
Effects of 3S2 Truck Loads on Top Surface of Continuous Bridge Decks

Ratio of Max Value of Stress of 3S2 to HS20-44						
Span Length	**Max Tensile Stress**			**Max Compressive Stress**		
(ft.)	***Longitudinal***	***Transverse***	***Shear***	***Longitudinal***	***Transverse***	***Shear***
20	0.912	0.722	1.588	0.719	0.962	2.229
30	1.150	0.707	1.266	0.577	0.896	1.145
45	1.739	1.059	0.870	0.705	1.006	0.975
60	1.599	1.168	0.970	0.711	0.950	0.996
75	1.247	1.284	1.232	0.746	1.025	1.504
90	1.356	1.324	1.348	1.092	1.062	1.295
105	1.385	1.332	1.335	0.813	1.104	1.411
120	1.430	1.371	1.370	0.997	1.093	1.384

Table 6
Effects of 3S2 Truck Loads on Bottom Surface of Continuous Bridge Decks

Ratio of Max Value of Stress of 3S2 to HS20-44						
Span Length	**Max Tensile Stress**			**Max Compressive Stress**		
(ft.)	***Longitudinal***	***Transverse***	***Shear***	***Longitudinal***	***Transverse***	***Shear***
20	0.719	0.962	2.229	0.912	0.722	1.588
30	0.577	0.896	1.145	1.150	0.707	1.266
45	0.705	1.006	0.975	1.739	1.059	0.870
60	0.711	0.950	0.996	1.599	1.168	0.970
75	0.746	1.025	1.504	1.247	1.284	1.232
90	1.092	1.062	1.295	1.356	1.324	1.348
105	0.813	1.104	1.411	1.385	1.332	1.335
120	0.997	1.093	1.384	1.430	1.371	1.370

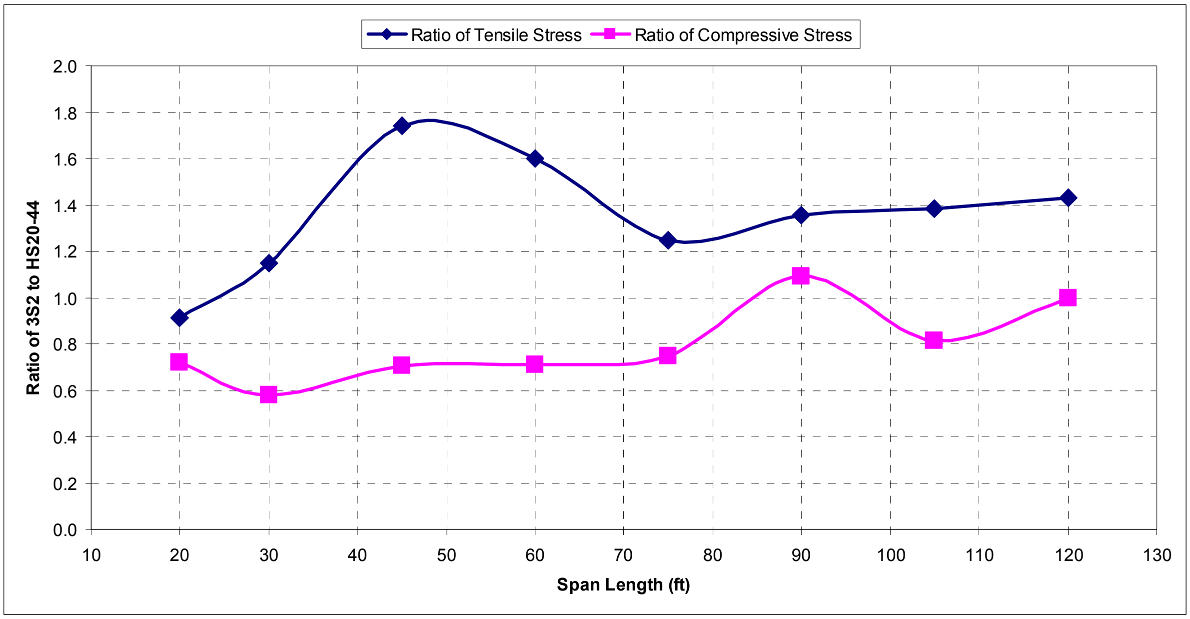

Figure 17
Effects on Longitudinal Stress Top Surface of Continuous Bridge Decks

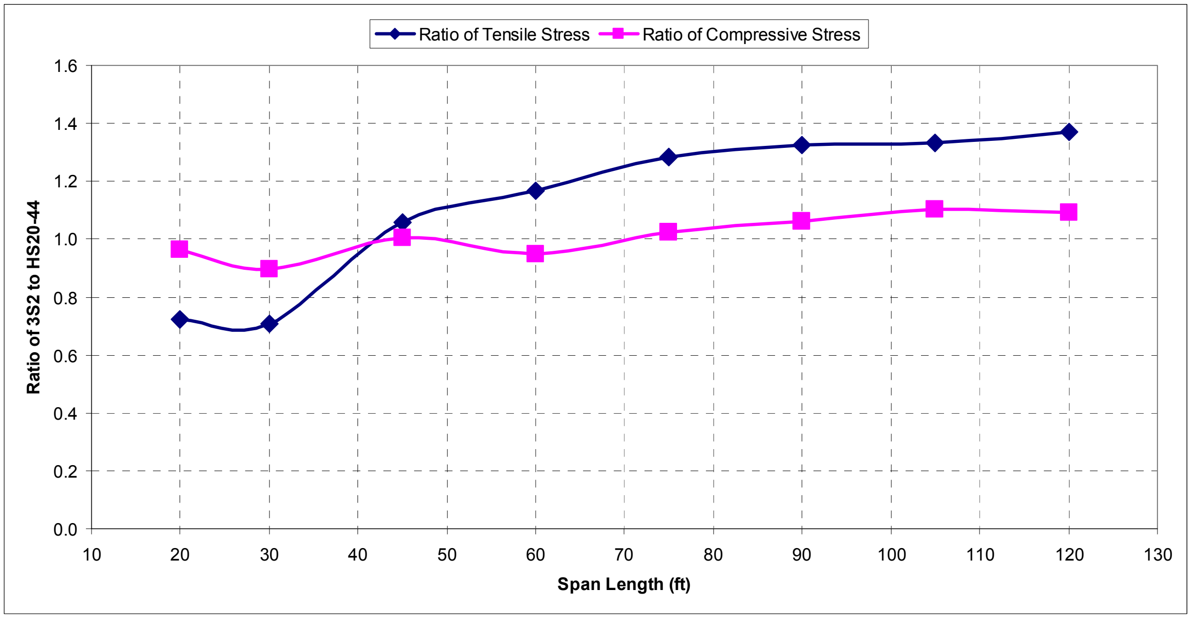

Figure 18
Effects on Transverse Stress Top Surface of Continuous Bridge Decks

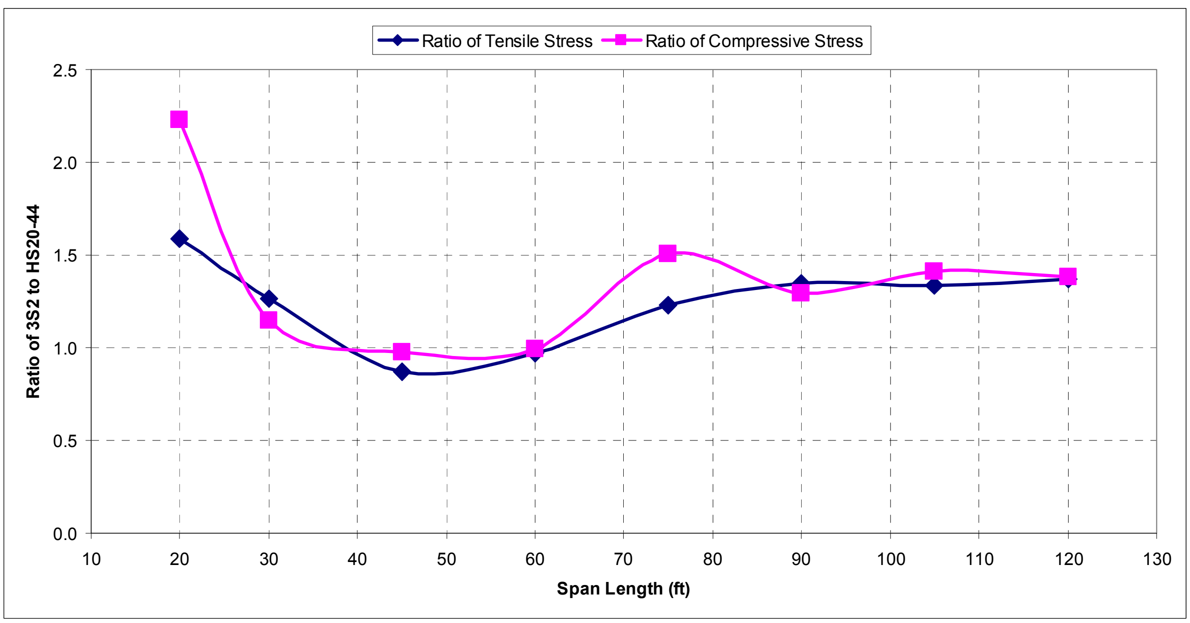

Figure 19
Effects on Shear Stress Top Surface of Continuous Bridge Decks

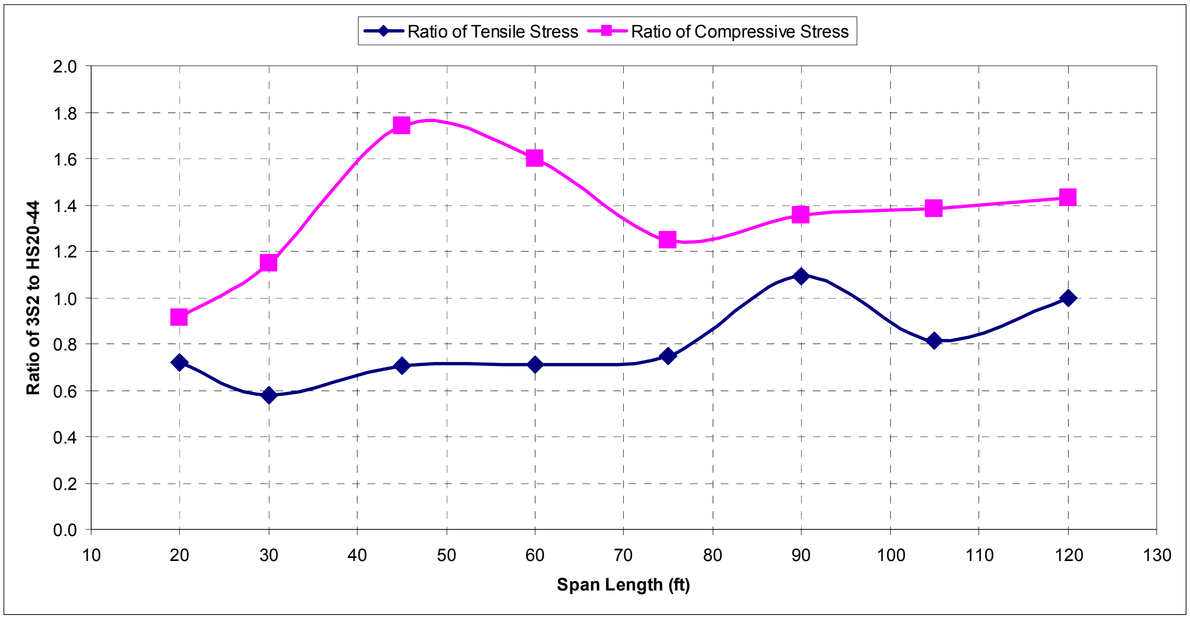

Figure 20
Effects on Longitudinal Stress Bottom Surface of Continuous Bridge Decks

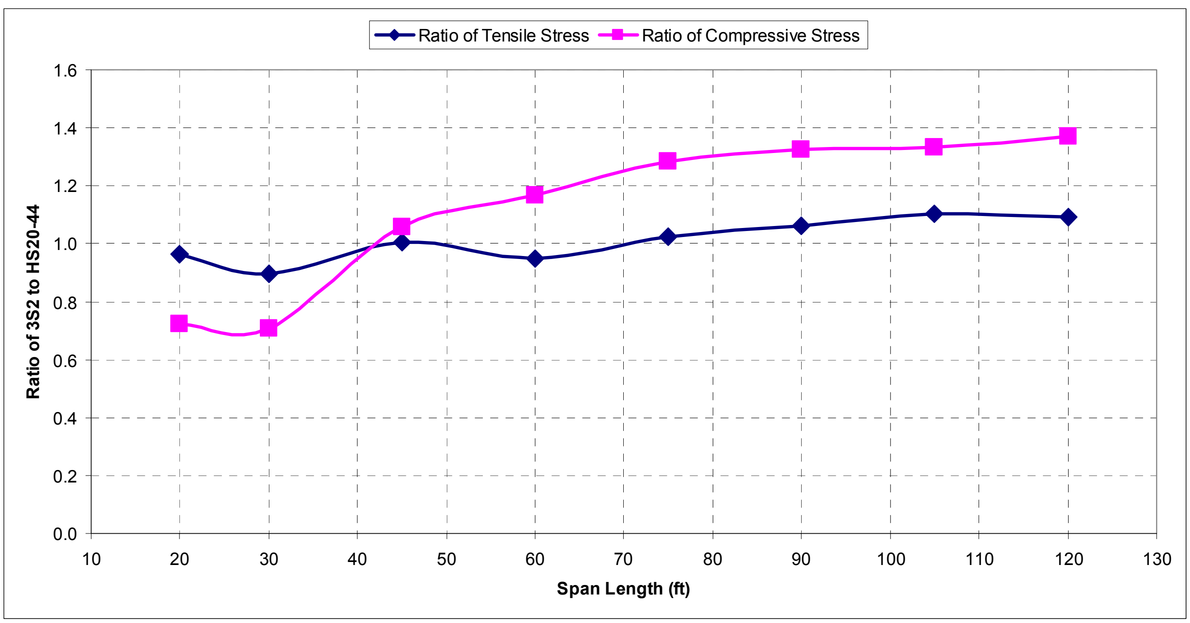

Figure 21
Effects on Transverse Stress Bottom Surface of Continuous Bridge Decks

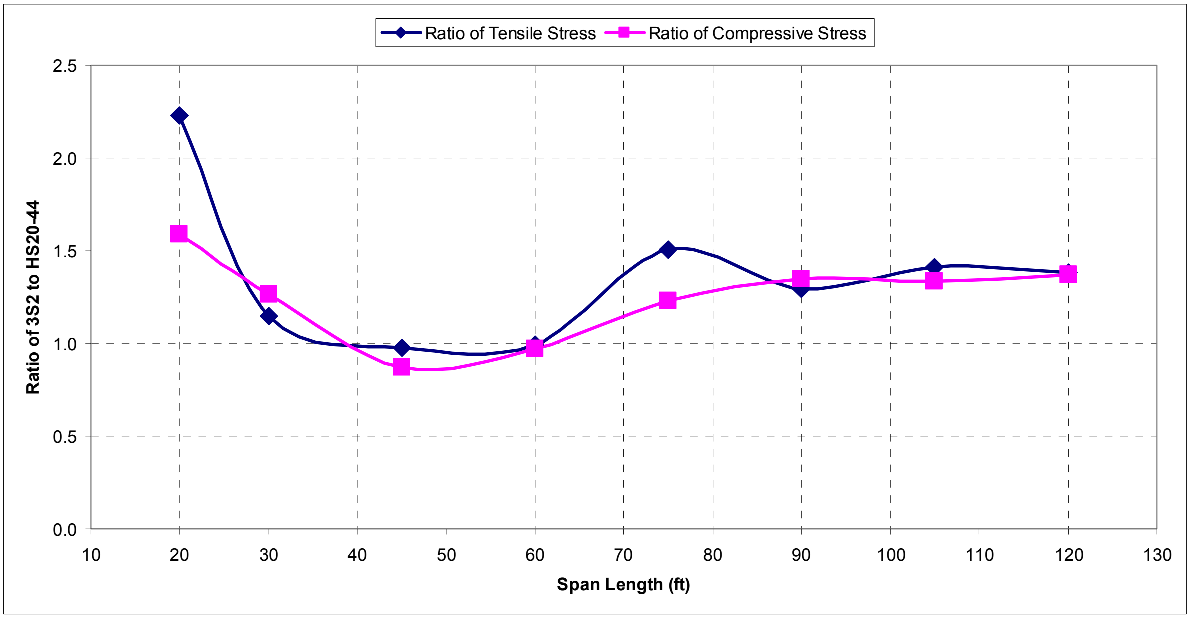

Figure 22
Effects on Shear Stress Bottom Surface of Continuous Bridge Decks

APPENDIX D
Results for Parish Bridges

Table 1
Bridges and Categories Considered for Analysis

Truck Hauling	Category	Design Load HS20-44	Total	Cost Per Trip	
				Based on \$140psf	Based on \$90psf
Timber	Beam Simple	166	166	\$1.6	\$1.05
	Continuous	1	1	\$7.3	\$4.7

Table 2
Simply Supported Bridges with Design Load HS20-44

Max Span Length (ft.)	Number of Bridges	Ratio 3S2/HS20-44		
		Moment	Shear	Deflection
15	5	1.13	1.22	1.29
20	155	1.22	1.10	1.37
40	1	1.07	0.98	1.04
50	5	1.00	1.08	0.96
Total	166			

Table 3
Continuous Bridges with Design Load HS20-44

Max Span Length (ft.)	Number of Bridges	Ratio 3S2/HS20-44		
		Positive Moment	Negative Moment	Shear
50	1	1.02	1.48	1.08

Table 4
Fatigue Cost Based on $140psf for Simply Supported Bridges with Design Load HS20-44

Main Span Length (ft.)	**Total Span Length (ft.)**	**Number of Bridges**	**Number of Bridges * Total Length**	***Ratio from Flexure Analysis***	**% of life**	**Cost per Trip * # of Bridges * Total Length**
15	30	1	30	***1.13***	3.16E-06	$11.95
15	35	3	105	***1.13***	3.16E-06	$48.81
20	35	1	35	***1.22***	3.93E-06	$20.23
20	40	3	120	***1.22***	3.93E-06	$79.25
20	50	1	50	***1.22***	3.93E-06	$41.28
20	56	22	1232	***1.22***	3.93E-06	$1,139.13
20	60	11	660	***1.22***	3.93E-06	$653.84
20	75	21	1575	***1.22***	3.93E-06	$1,950.37
20	80	18	1440	***1.22***	3.93E-06	$1,902.08
20	95	19	1805	***1.22***	3.93E-06	$2,831.24
20	100	13	1300	***1.22***	3.93E-06	$2,146.44
20	115	15	1725	***1.22***	3.93E-06	$3,275.38
20	120	4	480	***1.22***	3.93E-06	$951.04
20	135	14	1890	***1.22***	3.93E-06	$4,212.80
20	140	3	420	***1.22***	3.93E-06	$970.85
20	150	2	300	***1.22***	3.93E-06	$743.00
20	155	1	155	***1.22***	3.93E-06	$396.68
20	160	1	160	***1.22***	3.93E-06	$422.68
20	170	4	680	***1.22***	3.93E-06	$1,908.68
20	190	2	380	***1.22***	3.93E-06	$1,192.10
20	220	1	220	***1.22***	3.93E-06	$799.14
40	195	1	195	***1.07***	2.67E-06	$426.02
50	200	1	200	***1.00***	0.00E+00	$0.00
50	240	3	720	***1.00***	0.00E+00	$0.00
50	250	1	250	***1.00***	0.00E+00	$0.00
Sum			16127			$26,122.99
weighted average cost per trip						$1.62

Table 4 Cont.

Main Span Length (ft.)	Total Span Length (ft.)	Number of Bridges	Number of Bridges * Total Length	***Ratio from Shear Analysis***	% of life	Cost per Trip * # of Bridges * Total Length
15	30	1	30	***1.22***	3.98E-06	$15.04
15	35	3	105	***1.22***	3.98E-06	$61.43
20	35	1	35	***1.10***	2.92E-06	$15.01
20	40	3	120	***1.10***	2.92E-06	$58.81
20	50	1	50	***1.10***	2.92E-06	$30.63
20	56	22	1232	***1.10***	2.92E-06	$845.32
20	60	11	660	***1.10***	2.92E-06	$485.20
20	75	21	1575	***1.10***	2.92E-06	$1,447.33
20	80	18	1440	***1.10***	2.92E-06	$1,411.49
20	95	19	1805	***1.10***	2.92E-06	$2,101.00
20	100	13	1300	***1.10***	2.92E-06	$1,592.82
20	115	15	1725	***1.10***	2.92E-06	$2,430.59
20	120	4	480	***1.10***	2.92E-06	$705.74
20	135	14	1890	***1.10***	2.92E-06	$3,126.22
20	140	3	420	***1.10***	2.92E-06	$720.45
20	150	2	300	***1.10***	2.92E-06	$551.36
20	155	1	155	***1.10***	2.92E-06	$294.37
20	160	1	160	***1.10***	2.92E-06	$313.66
20	170	4	680	***1.10***	2.92E-06	$1,416.39
20	190	2	380	***1.10***	2.92E-06	$884.63
20	220	1	220	***1.10***	2.92E-06	$593.02
40	195	1	195	***0.98***	0.00E+00	$0.00
50	200	1	200	***1.08***	2.76E-06	$463.85
50	240	3	720	***1.08***	2.76E-06	$2,003.83
50	250	1	250	***1.08***	2.76E-06	$724.77
Sum			16127			$22,292.96
weighted average cost per trip						$1.38

Table 5
Fatigue Cost Based on $140psf Results from Flexural Analysis for Continuous Bridges with Design Load HS20-44

Span Length (ft.)	**Total Length (ft.)**	**Number of Bridges**	**Number of Bridges * Total Length**	***Ratio from Flexure Analysis***		**% of Life**		**Cost per Trip * # of Bridges * Total Length**	
				Positive Moment	***Negative Moment***	Positive Moment	Negative Moment	Positive Moment	Negative Moment
50	245	1	245	***1.02***	***1.48***	2.33E-06	7.13E-06	$586.46	$1,798.27
Sum			245					$586.46	$1,798.27
weighted average cost per trip								$2.39	$7.34

Table 6
Fatigue Cost Based on $140psf Results from Shear Analysis for Continuous Bridges with Design Load HS20-44

Span Length (ft.)	**Total Length (ft.)**	**Number of Bridges**	**Number of Bridges * Total Length**	***Ratio from Shear Analysis***	**% of Life**	**Cost per Trip * # of Bridges * Total Length**
50	245	1	245	***1.08***	2.76E-06	$696.07
Sum			245			$696.07
weighted average cost per trip						$2.84

Table 7
Fatigue Cost Based on $90psf for Simply Supported Bridges with Design Load HS20-44

Main Span Length (ft)	Total Span Length (ft)	Number of Bridges	Number of Bridges * Total Length	***Ratio from Flexure Analysis***	% of life	Cost per Trip	Cost per Trip * # of Bridges * Total Length
15	30	1	30	***1.13***	3.16E-06	$0.26	$7.68
15	35	3	105	***1.13***	3.16E-06	$0.30	$31.38
20	35	1	35	***1.22***	3.93E-06	$0.37	$13.00
20	40	3	120	***1.22***	3.93E-06	$0.42	$50.95
20	50	1	50	***1.22***	3.93E-06	$0.53	$26.54
20	56	22	1232	***1.22***	3.93E-06	$0.59	$732.30
20	60	11	660	***1.22***	3.93E-06	$0.64	$420.32
20	75	21	1575	***1.22***	3.93E-06	$0.80	$1,253.81
20	80	18	1440	***1.22***	3.93E-06	$0.85	$1,222.76
20	95	19	1805	***1.22***	3.93E-06	$1.01	$1,820.08
20	100	13	1300	***1.22***	3.93E-06	$1.06	$1,379.85
20	115	15	1725	***1.22***	3.93E-06	$1.22	$2,105.60
20	120	4	480	***1.22***	3.93E-06	$1.27	$611.38
20	135	14	1890	***1.22***	3.93E-06	$1.43	$2,708.23
20	140	3	420	***1.22***	3.93E-06	$1.49	$624.12
20	150	2	300	***1.22***	3.93E-06	$1.59	$477.64
20	155	1	155	***1.22***	3.93E-06	$1.65	$255.01
20	160	1	160	***1.22***	3.93E-06	$1.70	$271.73
20	170	4	680	***1.22***	3.93E-06	$1.80	$1,227.01
20	190	2	380	***1.22***	3.93E-06	$2.02	$766.35
20	220	1	220	***1.22***	3.93E-06	$2.34	$513.73
40	195	1	195	***1.07***	2.67E-06	$1.40	$273.87
50	200	1	200	***1.00***	0.00E+00	$0.00	$0.00
50	240	3	720	***1.00***	0.00E+00	$0.00	$0.00
50	250	1	250	***1.00***	0.00E+00	$0.00	$0.00
Sum			**16127**				**$16,793.35**
	weighted average cost per trip						**$1.04**

Table 7 Cont'd

Main Span Length (ft)	Total Span Length (ft)	Number of Bridges	Number of Bridges * Total Length	***Ratio from Shear Analysis***	% of life	Cost per Trip	Cost per Trip * # of Bridges * Total Length
15	30	1	30	***1.22***	3.98E-06	$0.32	$9.67
15	35	3	105	***1.22***	3.98E-06	$0.38	$39.49
20	35	1	35	***1.10***	2.92E-06	$0.28	$9.65
20	40	3	120	***1.10***	2.92E-06	$0.32	$37.81
20	50	1	50	***1.10***	2.92E-06	$0.39	$19.69
20	56	22	1232	***1.10***	2.92E-06	$0.44	$543.42
20	60	11	660	***1.10***	2.92E-06	$0.47	$311.91
20	75	21	1575	***1.10***	2.92E-06	$0.59	$930.42
20	80	18	1440	***1.10***	2.92E-06	$0.63	$907.38
20	95	19	1805	***1.10***	2.92E-06	$0.75	$1,350.64
20	100	13	1300	***1.10***	2.92E-06	$0.79	$1,023.96
20	115	15	1725	***1.10***	2.92E-06	$0.91	$1,562.52
20	120	4	480	***1.10***	2.92E-06	$0.95	$453.69
20	135	14	1890	***1.10***	2.92E-06	$1.06	$2,009.72
20	140	3	420	***1.10***	2.92E-06	$1.10	$463.14
20	150	2	300	***1.10***	2.92E-06	$1.18	$354.45
20	155	1	155	***1.10***	2.92E-06	$1.22	$189.24
20	160	1	160	***1.10***	2.92E-06	$1.26	$201.64
20	170	4	680	***1.10***	2.92E-06	$1.34	$910.54
20	190	2	380	***1.10***	2.92E-06	$1.50	$568.69
20	220	1	220	***1.10***	2.92E-06	$1.73	$381.23
40	195	1	195	***0.98***	0.00E+00	$0.00	$0.00
50	200	1	200	***1.08***	2.76E-06	$1.49	$298.19
50	240	3	720	***1.08***	2.76E-06	$1.79	$1,288.18
50	250	1	250	***1.08***	2.76E-06	$1.86	$465.92
Sum			**16127**				**$14,331.19**
	weighted average cost per trip						**$0.89**

Table 8
Fatigue Cost Based on $90psf Results from Flexural Analysis for Continuous Bridges with Design Load HS20-44

Span Length (ft)	Total Length (ft)	Number of Bridges	Number of Bridges * Total Length	Ratio from Flexure Analysis		% of Life		Cost per Trip		Cost per Trip * # of Bridges * Total Length	
				Positive Moment	Negative Moment	Positive Moment	Negative Moment	Positive Moment	Negative Moment	Positive Moment	Negative Moment
50	245	1	245	1.02	1.48	2.33E-06	7.13E-06	$2	$5	$377.01	$1,156.03
Sum			**245**							**$377.01**	**$1,156.03**
	weighted average cost per trip									**$1.54**	**$4.72**

Table 9
Fatigue Cost Based on $90psf Results from Shear Analysis for Continuous Bridges with Design Load HS20-44

Span Length (ft)	Total Length (ft)	Number of Bridges	Number of Bridges * Total Length	Ratio from Shear Analysis	% of Life	Cost per Trip	Cost per Trip * # of Bridges * Total Length
50	245	1	245	1.08	2.76E-06	$2	$447.47
Sum			**245**				**$447.47**
	weighted average cost per trip						**$1.83**

APPENDIX E
Fatigue Cost Study for State Bridges

Procedure used in calculating the weighted average cost per trip presented in tables 1 through 5, Appendix E.

1. Multiply the value of the cost per trip by the number of bridges of certain span length to get the cost per trip via all certain span length bridges.
2. Multiply the value of the cost per trip by the number of main spans to get the cost per trip via all certain span length.
3. Multiply the value of the cost per trip by the number of bridges of certain span length by the number of main spans to get the total cost via all certain span length bridges.
4. Multiply the values of the number of bridges and number of main spans.
5. Sum the values of the number of bridges, number of main spans, and the value of step 4.
6. Sum the values obtained from step 1, step 2 and step 3.
7. Divide results obtained from step 6 by the values obtained from 4 and 5, respectively to find the weighted average cost per trip.

Table 1
Summary Fatigue Cost for 3S2 trucks on Louisiana State bridges

Bridge Support Condition	**Design Load**	**Cost per Trip**	
		Based on $3M	**Based on $90psf**
Simple	H15	$39	$8.5
Simple	HS20-44	$11.5	$5.75
Continuous	HS20-44	$16	$8.9

Table 2
Fatigue Cost Based on $3M for Simply Supported Bridges with Design Load H15

3S2/H15								
Span Length	**Number of Main Spans**	**Number of Bridges**	***Ratio from Flexure Analysis***	**% of Life**	**Cost per Trip * # of Bridges**	**Cost per Trip * # of Main Spans**	**# of Main Spans*#of bridges**	**Cost per Trip*# of Main Spans* # of bridges**
20 ft. or Shorter	266	159	***1.62***	9.32E-06	$4,445	$7,436	980	$27,396
25 ft.	51	17	***1.70***	1.08E-05	$552	$1,655	90	$2,921
30 ft.	15	5	***1.79***	1.25E-05	$187	$561	17	$636
35 ft.	5	1	***1.82***	1.33E-05	$40	$199	5	$199
40 ft.	9	1	***1.85***	1.39E-05	$42	$375	9	$375
46 ft.	14	1	***1.87***	1.44E-05	$43	$606	14	$606
50 ft.	8	1	***1.89***	1.47E-05	$44	$353	8	$353
56 ft.	7	1	***1.91***	1.52E-05	$45	$318	7	$318
75 ft.	11	2	***2.07***	1.94E-05	$117	$642	11	$642
80 ft.	4	2	***2.07***	1.95E-05	$117	$234	8	$468
Sum	390	190			$5,632	$12,380	1149	$33,914
weighted average cost per trip					$29.64	$31.74		$29.52

Table 2 Cont.
Fatigue Cost Based on $3M for Simply Supported Bridges with Design Load H15

3S2/H15								
Span Length	**Number of Main Spans**	**Number of Bridges**	***Ratio from Shear Analysis***	**% of Life**	**Cost per Trip * # of Bridges**	**Cost per Trip * # of Main Spans**	**# of Main Spans*#of bridges**	**Cost per Trip*# of Main Spans* # of bridges**
20 ft. or Shorter	266	159	***1.77***	1.21E-05	$5,772	$9,657	980	$35,578
25 ft.	51	17	***1.82***	1.33E-05	$677	$2,030	90	$3,583
30 ft.	15	5	***1.85***	1.39E-05	$209	$627	17	$711
35 ft.	5	1	***1.86***	1.40E-05	$42	$210	5	$210
40 ft.	9	1	***1.86***	1.40E-05	$42	$378	9	$378
46 ft.	14	1	***1.95***	1.63E-05	$49	$683	14	$683
50 ft.	8	1	***2.00***	1.76E-05	$53	$423	8	$423
56 ft.	7	1	***2.06***	1.93E-05	$58	$405	7	$405
75 ft.	11	2	***2.08***	1.98E-05	$119	$653	11	$653
80 ft.	4	2	***2.07***	1.93E-05	$116	$232	8	$464
Sum	390	190			$7,136	$15,298.52	1149	$43,088
weighted average cost per trip					$37.56	$39.23		$37.50

Table 3
Fatigue Cost Based on $3M for Simply Supported Bridges with Design Load HS20-44

3S2/HS20-44								
Span Length	**Number of Main Spans**	**Number of Bridges**	***Ratio from Flexure Analysis***	**% of Life**	**Cost per Trip * # of Bridges**	**Cost per Trip * # of Main Spans**	**# of Main Spans*#of bridges**	**Cost per Trip*# of Main Spans* # of bridges**
20 ft. or Shorter	428	635	***1.22***	3.93E-06	$7,489	$5,048	4535	$53,484
25 ft.	100	30	***1.23***	4.09E-06	$369	$1,228	227	$2,788
30 ft.	3	1	***1.17***	3.52E-06	$11	$32	3	$32
35 ft.	18	7	***1.12***	3.10E-06	$65	$167	27	$251
40 ft.	67	15	***1.07***	2.67E-06	$120	$536	103	$824
46 ft.	86	15	***1.02***	2.36E-06	$106	$608	121	$856
50 ft.	79	16	***1.00***	0.00E+00	$0	$0	0	$0
56 ft.	33	3	***0.98***	0.00E+00	$0	$0	0	$0
60 ft.	57	13	***1.03***	2.36E-06	$92	$405	93	$660
66 ft.	20	4	***1.08***	2.74E-06	$33	$165	27	$222
70 ft.	20	17	***1.11***	2.97E-06	$151	$178	68	$605
75 ft.	15	7	***1.14***	3.22E-06	$68	$145	32	$309
80 ft.	11	2	***1.16***	3.45E-06	$21	$114	11	$114
85 ft.	43	5	***1.19***	3.66E-06	$55	$472	43	$472
90 ft.	12	5	***1.21***	3.84E-06	$58	$138	19	$219
95 ft.	12	4	***1.22***	4.01E-06	$48	$145	16	$193
100 ft.	53	6	***1.24***	4.17E-06	$75	$663	53	$663
105 ft.	5	4	***1.25***	4.31E-06	$52	$65	10	$129
110 ft.	8	1	***1.27***	4.44E-06	$13	$107	8	$107
115 ft.	4	1	***1.28***	4.56E-06	$14	$55	4	$55
120 ft.	45	1	***1.29***	4.67E-06	$14	$631	45	$631
Sum	1119	792			$8,852	$10,899	5445	$62,613
weighted average cost per trip					$11.18	$9.74		$11.50

Table 3 Cont.
Fatigue Cost Based on $3M for Simply Supported Bridges with Design Load HS20-44

3S2/HS20-44								
Span Length	**Number of Main Spans**	**Number of Bridges**	***Ratio from Shear Analysis***	**% of Life**	**Cost per Trip * # of Bridges**	**Cost per Trip * # of Main Spans**	**# of Main Spans*#of bridges**	**Cost per Trip*# of Main Spans* # of bridges**
20 ft. or Shorter	428	635	***1.10***	2.89E-06	$5,474	$3,664	4535	$39,275
25 ft.	100	30	***1.05***	2.55E-06	$230	$766	227	$1,738
30 ft.	3	1	***1.02***	2.30E-06	$7	$21	3	$21
35 ft.	18	7	***0.98***	0.00E+00	$0	$0	0	$0
40 ft.	57	14	***0.98***	0.00E+00	$0	$0	0	$0
46 ft.	86	15	***1.04***	2.44E-06	$110	$629	121	$886
50 ft.	79	16	***1.08***	2.74E-06	$132	$650	128	$1,054
56 ft.	33	3	***1.13***	3.18E-06	$29	$315	33	$315
60 ft.	57	13	***1.16***	3.43E-06	$134	$587	93	$958
66 ft.	20	4	***1.20***	3.76E-06	$45	$226	27	$305
70 ft.	20	17	***1.22***	3.95E-06	$201	$237	68	$806
75 ft.	15	7	***1.24***	4.16E-06	$87	$187	32	$400
80 ft.	11	2	***1.26***	4.35E-06	$26	$144	11	$144
85 ft.	43	5	***1.27***	4.52E-06	$68	$583	43	$583
90 ft.	12	5	***1.29***	4.67E-06	$70	$168	19	$266
95 ft.	12	4	***1.30***	4.81E-06	$58	$173	16	$231
100 ft.	53	6	***1.31***	4.93E-06	$89	$785	53	$785
105 ft.	5	4	***1.32***	5.05E-06	$61	$76	10	$151
110 ft.	8	1	***1.33***	5.15E-06	$15	$124	8	$124
115 ft.	4	1	***1.34***	5.24E-06	$16	$63	4	$63
120 ft.	45	1	***1.34***	5.33E-06	$16	$720	45	$720
Sum	1109	791			$6,867	$10,117	5476	$48,823
weighted average cost per trip					$8.68	$9.12		$8.92

Table 4 Fatigue

Fatigue Cost Based on $3M Results from Flexural Analysis for Continuous Bridges with Design Load HS20-44

Sum Volume: # of Bridges								
3S2/HS20-44								
Span Length	**Number of Main Spans**	**Number of Bridges**	***Ratio from Flexure Analysis***		**% of Life**		**Cost per Trip* # of Bridges**	
			Positive Moment	***Negative Moment***	**Positive Moment**	**Negative Moment**	**Positive Moment**	**Negative Moment**
20 or Shorter	15	3	***1.28***	***0.98***	4.54E-06	0.00E+00	$41	$0
45 ft.	19	2	***1.05***	***1.56***	2.52E-06	8.25E-06	$15	$49
50 ft.	125	14	***1.02***	***1.48***	2.33E-06	7.13E-06	$98	$300
55 ft.	3	1	***1.00***	***1.41***	0.00E+00	6.17E-06	$0	$18
60 ft.	7	4	***1.02***	***1.35***	2.35E-06	5.34E-06	$28	$64
65 ft.	25	6	***1.07***	***1.28***	2.69E-06	4.65E-06	$48	$84
70 ft.	71	15	***1.10***	***1.23***	2.92E-06	4.06E-06	$131	$183
75 ft.	87	10	***1.13***	***1.24***	3.13E-06	4.13E-06	$94	$124
80 ft.	11	2	***1.15***	***1.27***	3.32E-06	4.46E-06	$20	$27
85 ft.	9	5	***1.17***	***1.29***	3.50E-06	4.75E-06	$53	$71
90 ft.	37	20	***1.19***	***1.32***	3.67E-06	5.01E-06	$198	$270
95 ft.	36	3	***1.20***	***1.34***	3.82E-06	5.23E-06	$34	$47
100 ft.	85	13	***1.22***	***1.35***	3.97E-06	5.42E-06	$155	$211
105 ft.	47	20	***1.23***	***1.40***	4.10E-06	6.08E-06	$246	$365
110 ft.	19	2	***1.24***	***1.38***	4.22E-06	5.73E-06	$25	$34
120 ft.	20	2	***1.27***	***1.40***	4.45E-06	5.98E-06	$27	$36
125 ft.	15	4	***1.28***	***1.41***	4.55E-06	6.09E-06	$55	$73
130 ft.	8	2	***1.28***	***1.41***	4.64E-06	6.18E-06	$28	$37
Sum	639	128					$1,296	$1,994
Weighted Average Cost per Trip							$10.12	$15.58

Table 4 Cont.
Fatigue Cost Based on $3M Results from Flexural Analysis for Continuous Bridges with Design Load HS20-44

Sum Volume: # of Main Spans								
3S2/HS20-44								
Span Length	**Number of Main Spans**	**Number of Bridges**	***Ratio from Flexure Analysis***		**% of Life**		**Cost per Trip* # of Main Spans**	
			Positive Moment	***Negative Moment***	**Positive Moment**	**Negative Moment**	**Positive Moment**	**Negative Moment**
20 or Shorter	15	3	***1.28***	***0.98***	4.54E-06	0.00E+00	$204	$0
45 ft.	19	2	***1.05***	***1.56***	2.52E-06	8.25E-06	$144	$470
50 ft.	125	14	***1.02***	***1.48***	2.33E-06	7.13E-06	$872	$2,675
55 ft.	3	1	***1.00***	***1.41***	0.00E+00	6.17E-06	$0	$55
60 ft.	7	4	***1.02***	***1.35***	2.35E-06	5.34E-06	$49	$112
65 ft.	25	6	***1.07***	***1.28***	2.69E-06	4.65E-06	$202	$349
70 ft.	71	15	***1.10***	***1.23***	2.92E-06	4.06E-06	$622	$866
75 ft.	87	10	***1.13***	***1.24***	3.13E-06	4.13E-06	$817	$1,078
80 ft.	11	2	***1.15***	***1.27***	3.32E-06	4.46E-06	$110	$147
85 ft.	9	5	***1.17***	***1.29***	3.50E-06	4.75E-06	$95	$128
90 ft.	37	20	***1.19***	***1.32***	3.67E-06	5.01E-06	$363	$496
95 ft.	36	3	***1.20***	***1.34***	3.82E-06	5.23E-06	$413	$564
100 ft.	85	13	***1.22***	***1.35***	3.97E-06	5.42E-06	$1,012	$1,381
105 ft.	47	20	***1.23***	***1.40***	4.10E-06	6.08E-06	$578	$857
110 ft.	19	2	***1.24***	***1.38***	4.22E-06	5.73E-06	$241	$327
120 ft.	20	2	***1.27***	***1.40***	4.45E-06	5.98E-06	$267	$359
125 ft.	15	4	***1.28***	***1.41***	4.55E-06	6.09E-06	$205	$274
130 ft.	8	2	***1.28***	***1.41***	4.64E-06	6.18E-06	$111	$148
Sum	639	128					$6,305	$10,286
Weighted Average Cost per Trip							$9.87	$16.10

Table 4 Cont.
Fatigue Cost Based on $3M Results from Flexural Analysis for Continuous Bridges with Design Load HS20-44

Sum Volume: # of Main Spans*# of Bridges									
3S2/HS20-44									
Span Length	**Number of Main Spans**	**Number of Bridges**	***Ratio from Flexure Analysis***		**% of Life**		**# of Main Spans * # of Bridges**	**Cost per Trip*# of Main Spans*# of bridges**	
			Positive Moment	***Negative Moment***	**Positive Moment**	**Negative Moment**		**Positive Moment**	**Negative Moment**
20 or Shorter	15	3	***1.28***	***0.98***	4.54E-06	0.00E+00	45	$613	$0
45 ft.	19	2	***1.05***	***1.56***	2.52E-06	8.25E-06	19	$144	$470
50 ft.	125	14	***1.02***	***1.48***	2.33E-06	7.13E-06	140	$977	$2,996
55 ft.	3	1	***1.00***	***1.41***	0.00E+00	6.17E-06	3	$0	$55
60 ft.	7	4	***1.02***	***1.35***	2.35E-06	5.34E-06	14	$99	$224
65 ft.	25	6	***1.07***	***1.28***	2.69E-06	4.65E-06	50	$404	$697
70 ft.	71	15	***1.10***	***1.23***	2.92E-06	4.06E-06	126	$1,104	$1,536
75 ft.	87	10	***1.13***	***1.24***	3.13E-06	4.13E-06	120	$1,127	$1,486
80 ft.	11	2	***1.15***	***1.27***	3.32E-06	4.46E-06	11	$110	$147
85 ft.	9	5	***1.17***	***1.29***	3.50E-06	4.75E-06	16	$168	$228
90 ft.	37	20	***1.19***	***1.32***	3.67E-06	5.01E-06	93	$1,024	$1,397
95 ft.	36	3	***1.20***	***1.34***	3.82E-06	5.23E-06	36	$413	$564
100 ft.	85	13	***1.22***	***1.35***	3.97E-06	5.42E-06	102	$1,214	$1,658
105 ft.	47	20	***1.23***	***1.40***	4.10E-06	6.08E-06	83	$1,021	$1,513
110 ft.	19	2	***1.24***	***1.38***	4.22E-06	5.73E-06	19	$241	$327
120 ft.	20	2	***1.27***	***1.40***	4.45E-06	5.98E-06	40	$534	$718
125 ft.	15	4	***1.28***	***1.41***	4.55E-06	6.09E-06	19	$259	$347
130 ft.	8	2	***1.28***	***1.41***	4.64E-06	6.18E-06	8	$111	$148
Sum	639	128					944	$9,562	$14,513
Weighted Average Cost per Trip								$10.13	$15.37

Table 5
Fatigue Cost Based on $3M Results from Shear Analysis for Continuous Bridges with Design Load HS20-44

3S2/HS20-44								
Span Length	**Number of Main Spans**	**Number of Bridges**	***Ratio from Shear Analysis***	**% of Life**	**Cost per Trip* # of Bridges**	**Cost per Trip* # of Main Spans**	**# of Main Spans * # of Bridges**	**Cost per Trip*# of Main Spans*# of bridges**
20 or Shorter	15	3	***1.07***	2.69E-06	$24	$121	45	$363
45 ft.	19	2	***1.04***	2.43E-06	$15	$139	19	$139
50 ft.	125	14	***1.08***	2.78E-06	$117	$1,043	140	$1,168
55 ft.	3	1	***1.14***	3.22E-06	$10	$29	3	$29
60 ft.	7	4	***1.18***	3.62E-06	$43	$76	14	$152
65 ft.	25	6	***1.22***	3.97E-06	$71	$297	50	$595
70 ft.	71	15	***1.25***	4.26E-06	$192	$908	126	$1,611
75 ft.	87	10	***1.27***	4.52E-06	$136	$1,180	120	$1,627
80 ft.	11	2	***1.29***	4.74E-06	$28	$157	11	$157
85 ft.	9	5	***1.31***	4.94E-06	$74	$133	16	$237
90 ft.	37	20	***1.33***	5.11E-06	$307	$568	93	$1,427
95 ft.	36	3	***1.34***	5.27E-06	$47	$569	36	$569
100 ft.	85	13	***1.35***	5.40E-06	$211	$1,378	102	$1,653
105 ft.	47	20	***1.36***	5.52E-06	$331	$779	83	$1,376
110 ft.	19	2	***1.37***	5.63E-06	$34	$321	19	$321
120 ft.	20	2	***1.38***	5.82E-06	$35	$349	40	$698
125 ft.	15	4	***1.39***	5.90E-06	$71	$266	19	$336
130 ft.	8	2	***1.40***	5.97E-06	$36	$143	8	$143
Sum	639	128			$1,782	$8,455	944	$12,601
Weighted Average Cost per Trip					$13.92	$13.23		$13.35

Table 6
Fatigue Cost Based on $90psf for Simply Supported Bridges with Design Load H15

3S2/H15								
Span Length	Number of Main Spans	Number of Bridges	Total Length (ft)	Total Length * # of Bridges	**Ratio from Flexure Analysis**	% of Life	Cost per Trip (Dollars)	Cost per Trip * # of Bridges * Total Length
20 ft or shorter	266	159	5320	19600	**1.62**	0.0000093	$134	$100,498
25 ft	51	17	1275	2250	**1.70**	0.0000108	$37	$11,719
30 ft	15	5	450	510	**1.79**	0.0000125	$15	$2,092
35 ft	5	1	175	175	**1.82**	0.0000133	$6	$1,100
40 ft	9	1	360	360	**1.85**	0.0000139	$14	$4,861
46 ft	14	1	644	644	**1.87**	0.0000144	$25	$16,148
50 ft	8	1	400	400	**1.89**	0.0000147	$16	$6,349
56 ft	7	1	392	392	**1.91**	0.0000152	$16	$6,292
75 ft	11	2	825	825	**2.07**	0.0000194	$43	$19,195
80 ft	4	2	320	640	**2.07**	0.0000195	$17	$10,785
Sum				**25796**				**$179,038**
			weighted average cost per trip					**$6.94**

Table 6 Cont'd.
Fatigue Cost Based on $90psf for Simply Supported Bridges with Design Load H15

3S2/H15								
Span Length	Number of Main Spans	Number of Bridges	Total Length (ft)	Total Length * # of Bridges	**Ratio from Shear Analysis**	% of Life	Cost per Trip (Dollars)	Cost per Trip * # of Bridges * Total Length
20 ft or shorter	266	159	5320	19600	**1.77**	0.0000121	$174	$130,511
25 ft	51	17	1275	2250	**1.82**	0.0000133	$46	$14,375
30 ft	15	5	450	510	**1.85**	0.0000139	$17	$2,338
35 ft	5	1	175	175	**1.86**	0.0000140	$7	$1,158
40 ft	9	1	360	360	**1.86**	0.0000140	$14	$4,901
46 ft	14	1	644	644	**1.95**	0.0000163	$28	$18,202
50 ft	8	1	400	400	**2.00**	0.0000176	$19	$7,618
56 ft	7	1	392	392	**2.06**	0.0000193	$20	$8,004
75 ft	11	2	825	825	**2.08**	0.0000198	$44	$19,531
80 ft	4	2	320	640	**2.07**	0.0000193	$17	$10,686
Sum				**25796**				**$217,324**
		weighted average cost per trip						**$8.42**

Table 7
Fatigue Cost Based on $90psf for Simply Supported Bridges with Design Load HS20-44

3S2/HS20-44								
Span Length	Number of Main Spans	Number of Bridges	Total Length (ft)	Total Length * # of Bridges	**Ratio from Flexure Analysis**	% of Life	Cost per Trip (Dollars)	Cost per Trip * # of Bridges * Total Length
20 ft or shorter	419	630	8380	90220	**1.22**	0.0000039	$89	$190,339
25 ft	100	30	2500	5675	**1.23**	0.0000041	$28	$15,885
30 ft	3	1	90	90	**1.17**	0.0000035	$1	$77
35 ft	18	7	630	945	**1.12**	0.0000031	$5	$1,157
40 ft	57	14	2280	3720	**1.07**	0.0000027	$16	$12,181
46 ft	86	15	3956	5566	**1.02**	0.0000024	$25	$19,974
50 ft	79	16	3950	6400	**1.00**	0.0000022	$24	$17,265
56 ft	33	3	1848	1848	**0.98**	0.0000000	$0	$0
60 ft	51	12	3060	5220	**1.03**	0.0000024	$20	$17,893
66 ft	20	4	1320	1782	**1.08**	0.0000027	$10	$7,836
70 ft	20	17	1400	4760	**1.11**	0.0000030	$11	$11,541
75 ft	15	7	1125	2400	**1.14**	0.0000032	$10	$7,341
80 ft	11	2	880	880	**1.16**	0.0000035	$8	$3,638
85 ft	43	5	3655	3655	**1.19**	0.0000037	$36	$34,466
90 ft	12	5	1080	1710	**1.21**	0.0000038	$11	$6,307
95 ft	12	4	1140	1520	**1.22**	0.0000040	$12	$8,022
100 ft	53	6	5300	5300	**1.24**	0.0000042	$60	$77,562
105 ft	5	4	525	1050	**1.25**	0.0000043	$6	$3,336
110 ft	8	1	880	880	**1.27**	0.0000044	$11	$9,285
115 ft	4	1	460	460	**1.28**	0.0000046	$6	$2,605
120 ft	45	1	5400	5400	**1.29**	0.0000047	$68	$367,746
Sum				**149481**				**$814,457**
		weighted average cost per trip						**$5.45**

Table 7 Cont'd.
Fatigue Cost Based on $90psf for Simply Supported Bridges with Design Load HS20-44

3S2/HS20-44								
Span Length	Number of Main Spans	Number of Bridges	Total Length (ft)	Total Length * # of Bridges	**Ratio from Shear Analysis**	% of Life	Cost per Trip (Dollars)	Cost per Trip * # of Bridges * Total Length
20 ft or shorter	419	630	8380	90220	**1.10**	0.0000029	$65	$139,770
25 ft	100	30	2500	5675	**1.05**	0.0000026	$17	$9,902
30 ft	3	1	90	90	**1.02**	0.0000023	$1	$50
35 ft	18	7	630	945	**0.98**	0.0000021	$0	$0
40 ft	57	14	2280	3720	**0.98**	0.0000021	$0	$0
46 ft	86	15	3956	5566	**1.04**	0.0000024	$26	$20,671
50 ft	79	16	3950	6400	**1.08**	0.0000027	$29	$21,380
56 ft	33	3	1848	1848	**1.13**	0.0000032	$16	$12,415
60 ft	51	12	3060	5220	**1.16**	0.0000034	$28	$25,984
66 ft	20	4	1320	1782	**1.20**	0.0000038	$13	$10,745
70 ft	20	17	1400	4760	**1.22**	0.0000040	$15	$15,368
75 ft	15	7	1125	2400	**1.24**	0.0000042	$13	$9,487
80 ft	11	2	880	880	**1.26**	0.0000044	$10	$4,589
85 ft	43	5	3655	3655	**1.27**	0.0000045	$45	$42,615
90 ft	12	5	1080	1710	**1.29**	0.0000047	$14	$7,667
95 ft	12	4	1140	1520	**1.30**	0.0000048	$15	$9,613
100 ft	53	6	5300	5300	**1.31**	0.0000049	$71	$91,802
105 ft	5	4	525	1050	**1.32**	0.0000050	$7	$3,906
110 ft	8	1	880	880	**1.33**	0.0000052	$12	$10,769
115 ft	4	1	460	460	**1.34**	0.0000052	$7	$2,996
120 ft	45	1	5400	5400	**1.34**	0.0000053	$78	$419,758
Sum				**149481**				**$859,487**
		weighted average cost per trip						**$5.75**

Table 8
Fatigue Cost Based on $90psf Results from Flexural Analysis for Continuous Bridges with Design Load HS20-44

3S2/HS20-44												
Span Length	Number of Main Spans	Number of Bridges	Total Length (ft)	Average Length of each Bridge	**Ratio from Flexure Analysis**		% of Life		Cost per Trip		Cost per Trip * Total Length	
					Positive Moment	**Negative Moment**	Positive Moment	Negative Moment	Positive Moment	Negative Moment	Positive Moment	Negative Moment
20 or Shorter	45	3	900	300.00	**1.28**	**0.98**	4.54E-06	2.04E-06	$4	$0	$3,313	$0
45 ft	19	2	740	370.00	**1.05**	**1.56**	2.52E-06	8.25E-06	$3	$8	$1,861	$6,096
50 ft	150	14	7511	536.50	**1.02**	**1.48**	2.33E-06	7.13E-06	$3	$10	$25,310	$77,608
55 ft	3	1	166	166.00	**1.00**	**1.41**	2.20E-06	6.17E-06	$1	$3	$163	$459
60 ft	14	4	708	177.00	**1.02**	**1.35**	2.35E-06	5.34E-06	$1	$3	$796	$1,806
65 ft	50	6	3300	550.00	**1.07**	**1.28**	2.69E-06	4.65E-06	$4	$7	$13,188	$22,780
70 ft	126	15	8030	535.33	**1.10**	**1.23**	2.92E-06	4.06E-06	$4	$6	$33,893	$47,176
75 ft	120	10	4765	476.50	**1.13**	**1.24**	3.13E-06	4.13E-06	$4	$5	$19,195	$25,312
80 ft	11	2	730	365.00	**1.15**	**1.27**	3.32E-06	4.46E-06	$3	$4	$2,392	$3,212
85 ft	16	5	1198	239.60	**1.17**	**1.29**	3.50E-06	4.75E-06	$2	$3	$2,716	$3,685
90 ft	85	18	6728	373.78	**1.19**	**1.32**	3.67E-06	5.01E-06	$4	$5	$24,919	$33,994
95 ft	36	3	3044	1014.67	**1.20**	**1.34**	3.82E-06	5.23E-06	$10	$14	$31,893	$43,575
100 ft	102	13	9251	711.62	**1.22**	**1.35**	3.97E-06	5.42E-06	$8	$10	$70,521	$96,282
105 ft	83	20	8036	401.80	**1.23**	**1.40**	4.10E-06	6.08E-06	$4	$7	$35,749	$52,972
110 ft	19	2	1711	855.50	**1.24**	**1.38**	4.22E-06	5.73E-06	$10	$13	$16,696	$22,660
120 ft	40	2	3570	1785.00	**1.27**	**1.40**	4.45E-06	5.98E-06	$21	$29	$76,504	$102,938
125 ft	19	4	1558	389.50	**1.28**	**1.41**	4.55E-06	6.09E-06	$5	$6	$7,449	$9,974
130 ft	8	2	864	432.00	**1.28**	**1.41**	4.64E-06	6.18E-06	$5	$7	$4,676	$6,229
Sum			**62,810**								**$371,231**	**$556,759**
	Weighted Average Cost per Trip										**$5.91**	**$8.86**

Table 9
Fatigue Cost Based on $90psf Results from Shear Analysis for Continuous Bridges with Design Load HS20-44

3S2/HS20-44								
Span Length	Number of Main Spans	Number of Bridges	Total Length (ft)	Average Length of each Bridge	Ratio from Shear Analysis	% of Life	Cost per Trip	Cost per Trip * Total Length
20 or Shorter	15	3	900	300.00	1.07	2.69E-06	$2	$1,961
45 ft	19	2	740	370.00	1.04	2.43E-06	$2	$1,800
50 ft	125	14	7511	536.50	1.08	2.78E-06	$4	$30,252
55 ft	3	1	166	166.00	1.14	3.22E-06	$1	$240
60 ft	7	4	708	177.00	1.18	3.62E-06	$2	$1,225
65 ft	25	6	3300	550.00	1.22	3.97E-06	$6	$19,431
70 ft	71	15	8030	535.33	1.25	4.26E-06	$6	$49,473
75 ft	87	10	4765	476.50	1.27	4.52E-06	$6	$27,710
80 ft	11	2	730	365.00	1.29	4.74E-06	$5	$3,413
85 ft	9	5	1198	239.60	1.31	4.94E-06	$3	$3,829
90 ft	33	18	6728	373.78	1.33	5.11E-06	$5	$34,722
95 ft	36	3	3044	1014.67	1.34	5.27E-06	$14	$43,921
100 ft	85	13	9251	711.62	1.35	5.40E-06	$10	$96,030
105 ft	47	20	8036	401.80	1.36	5.52E-06	$6	$48,159
110 ft	19	2	1711	855.50	1.37	5.63E-06	$13	$22,264
120 ft	20	2	3570	1785.00	1.38	5.82E-06	$28	$100,142
125 ft	15	4	1558	389.50	1.39	5.90E-06	$6	$9,669
130 ft	8	2	864	432.00	1.40	5.97E-06	$7	$6,021
Sum			**62,810**					**$500,261**
		Weighted Average Cost per Trip						**$7.96**

APPENDIX F
List of the Analysis Files

Table 1
Analysis Files of Influence Line Analysis of Simple Span Bridges

Influence Line Analysis - Simple Span Bridges	
Design Load	**File Name (Excel files .xls)**
H 15	Absolute Maximum Moment Shear & Deflection - 3-S2 vs H 15
HS20-44	Absolute Maximum Moment Shear & Deflection - 3-S2 vs HS 20-44
HS20-44	Scanned Calculation Max Moment Shear & Deflection - HS 20.pdf
3S2	Scanned Calculation Max Moment Shear & Deflection - 3 S2.pdf

Table 2
Analysis Files of Influence Line Analysis of Continuous Bridges – Moment

Span Length (ft.)	Influence Line Analysis - Continuous Bridges		
	GT Input files (.txt)	**Output Files (.gto)**	**EXCEL Files (.xls)**
20	IL - 3 Spans - 20 ft - Moment	IL - 3 Spans - 20 ft - Moment	IL - 3 Spans - 20 ft - Moment
30	IL - 3 Spans - 30 ft - Moment	IL - 3 Spans - 30 ft - Moment	IL - 3 Spans - 30 ft - Moment
40	IL - 3 Spans - 40 ft - Moment	IL - 3 Spans - 40 ft - Moment	IL - 3 Spans - 40 ft - Moment
45	IL - 3 Spans - 45 ft - Moment	IL - 3 Spans - 45 ft - Moment	IL - 3 Spans - 45 ft - Moment
50	IL - 3 Spans - 50 ft - Moment	IL - 3 Spans - 50 ft - Moment	IL - 3 Spans - 50 ft - Moment
55	IL - 3 Spans - 55 ft - Moment	IL - 3 Spans - 55 ft - Moment	IL - 3 Spans - 55 ft - Moment
60	IL - 3 Spans - 60 ft - Moment	IL - 3 Spans - 60 ft - Moment	IL - 3 Spans - 60 ft - Moment
65	IL - 3 Spans - 65 ft - Moment	IL - 3 Spans - 65 ft - Moment	IL - 3 Spans - 65 ft - Moment
70	IL - 3 Spans - 70 ft - Moment	IL - 3 Spans - 70 ft - Moment	IL - 3 Spans - 70 ft - Moment
75	IL - 3 Spans - 75 ft - Moment	IL - 3 Spans - 75 ft - Moment	IL - 3 Spans - 75 ft - Moment
80	IL - 3 Spans - 80 ft - Moment	IL - 3 Spans - 80 ft - Moment	IL - 3 Spans - 80 ft - Moment
85	IL - 3 Spans - 85 ft - Moment	IL - 3 Spans - 85 ft - Moment	IL - 3 Spans - 85 ft - Moment
90	IL - 3 Spans - 90 ft - Moment	IL - 3 Spans - 90 ft - Moment	IL - 3 Spans - 90 ft - Moment
95	IL - 3 Spans - 95 ft - Moment	IL - 3 Spans - 95 ft - Moment	IL - 3 Spans - 95 ft - Moment
100	IL - 3 Spans - 100 ft - Moment	IL - 3 Spans - 100 ft - Moment	IL - 3 Spans - 100 ft - Moment
105	IL - 3 Spans - 105 ft - Moment	IL - 3 Spans - 105 ft - Moment	IL - 3 Spans - 105 ft - Moment
110	IL - 3 Spans - 110 ft - Moment	IL - 3 Spans - 110 ft - Moment	IL - 3 Spans - 110 ft - Moment
115	IL - 3 Spans - 115 ft - Moment	IL - 3 Spans - 115 ft - Moment	IL - 3 Spans - 115 ft - Moment
120	IL - 3 Spans - 120 ft - Moment	IL - 3 Spans - 120 ft - Moment	IL - 3 Spans - 120 ft - Moment
125	IL - 3 Spans - 125 ft - Moment	IL - 3 Spans - 125 ft - Moment	IL - 3 Spans - 125 ft - Moment
130	IL - 3 Spans - 130 ft - Moment	IL - 3 Spans - 130 ft - Moment	IL - 3 Spans - 130 ft - Moment

Table 3
Analysis Files of Influence Line Analysis of Continuous Bridges – Shear

Span Length (ft)	Influence Line Analysis - Continuous Beam		
	GT Input files (.txt)	Output Files (.gto)	EXCEL Files (.xls)
20	IL - 3 Spans - 20 ft - Shear	IL - 3 Spans - 20 ft - Shear	IL - 3 Spans - 20 ft - Shear
30	IL - 3 Spans - 30 ft - Shear	IL - 3 Spans - 30 ft - Shear	IL - 3 Spans - 30 ft - Shear
40	IL - 3 Spans - 40 ft - Shear	IL - 3 Spans - 40 ft - Shear	IL - 3 Spans - 40 ft - Shear
45	IL - 3 Spans - 45 ft - Shear	IL - 3 Spans - 45 ft - Shear	IL - 3 Spans - 45 ft - Shear
50	IL - 3 Spans - 50 ft - Shear	IL - 3 Spans - 50 ft - Shear	IL - 3 Spans - 50 ft - Shear
55	IL - 3 Spans - 55 ft - Shear	IL - 3 Spans - 55 ft - Shear	IL - 3 Spans - 55 ft - Shear
60	IL - 3 Spans - 60 ft - Shear	IL - 3 Spans - 60 ft - Shear	IL - 3 Spans - 60 ft - Shear
65	IL - 3 Spans - 65 ft - Shear	IL - 3 Spans - 65 ft - Shear	IL - 3 Spans - 65 ft - Shear
70	IL - 3 Spans - 70 ft - Shear	IL - 3 Spans - 70 ft - Shear	IL - 3 Spans - 70 ft - Shear
75	IL - 3 Spans - 75 ft - Shear	IL - 3 Spans - 75 ft - Shear	IL - 3 Spans - 75 ft - Shear
80	IL - 3 Spans - 80 ft - Shear	IL - 3 Spans - 80 ft - Shear	IL - 3 Spans - 80 ft - Shear
85	IL - 3 Spans - 85 ft - Shear	IL - 3 Spans - 85 ft - Shear	IL - 3 Spans - 85 ft - Shear
90	IL - 3 Spans - 90 ft - Shear	IL - 3 Spans - 90 ft - Shear	IL - 3 Spans - 90 ft - Shear
95	IL - 3 Spans - 95 ft - Shear	IL - 3 Spans - 95 ft - Shear	IL - 3 Spans - 95 ft - Shear
100	IL - 3 Spans - 100 ft - Shear	IL - 3 Spans - 100 ft - Shear	IL - 3 Spans - 100 ft - Shear
105	IL - 3 Spans - 105 ft - Shear	IL - 3 Spans - 105 ft - Shear	IL - 3 Spans - 105 ft - Shear
110	IL - 3 Spans - 110 ft - Shear	IL - 3 Spans - 110 ft - Shear	IL - 3 Spans - 110 ft - Shear
115	IL - 3 Spans - 115 ft - Shear	IL - 3 Spans - 115 ft - Shear	IL - 3 Spans - 115 ft - Shear
120	IL - 3 Spans - 120 ft - Shear	IL - 3 Spans - 120 ft - Shear	IL - 3 Spans - 120 ft - Shear
125	IL - 3 Spans - 125 ft - Shear	IL - 3 Spans - 125 ft - Shear	IL - 3 Spans - 125 ft - Shear
130	IL - 3 Spans - 130 ft - Shear	IL - 3 Spans - 130 ft - Shear	IL - 3 Spans - 130 ft - Shear

Table 4
Analysis Files of Deck Analysis of Continuous Bridges – Positive Moment

Span Length (ft)	Deck Analysis - Positive Moment		
	GTSTRUDL Input files (.txt)	Output Files (.gto)	EXCEL Files (.xls)
20	3 Spans 20 ft -	middle - stress	
30	3 Spans 30 ft -	middle - stress	
45	3 Spans 45 ft -	middle - stress	3 Spans Stress Summary
60	3 Spans 60 ft -	middle – analysis - stress	-Positive
75	3 Spans 75 ft -	middle – analysis - stress	
90	3 Spans 90 ft - 3 S2	middle – analysis - stress	
	3 Spans 90 ft - HS 20	middle – analysis - stress	
105	3 Spans 105 ft -	middle – analysis - stress	
120	3 Spans 120 ft -	middle – analysis - stress	

Table 5
Analysis Files of Deck Analysis of Continuous Bridges – Negative Moment

Span Length (ft)	Deck Analysis - Negative Moment		
	GTSTRUDL Input files (.txt)	Output Files (.gto)	EXCEL Files (.xls)
20	3 Spans 20 ft - N	middle - stress	
30	3 Spans 30 ft - N	middle - stress	
45	3 Spans 45 ft - N	middle - stress	3 Spans Stress Summary
60	3 Spans 60 ft - N	middle - stress	-Negative
75	3 Spans 75 ft - N	middle - analysis - stress	
90	3 Spans 90 ft - N	middle - analysis - stress	
105	3 Spans 105 ft - N	middle - stress	
120	3 Spans 120 ft - N	middle - stress	