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YUKON RIVER BRIDGE, DECK STRAINS AND SURFACING ALTERNATIVES

by

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16. Abstract

A 2900-ft long bridge with six spans was built over the Yukon River in the state of Alaska in the 1970's. The bridge has a 30-ft roadway that carries vehicles, supports the pipeline, is on a 6% grade and is subjected to -50 degree winter temperatures. These conditions make selecting wearing surfaces a difficult decision. The bridge superstructure has an orthotropic steel deck that is overlaid with a temporary timber wearing surface. The timber deck consists of two layers of 3 by 12 boards and is supported by two 61-inch wide by 163-inch deep box girders. This study focused on predicting strain levels of possible alternative wearing surfaces.

Static strains were measured in the steel deck for several trucks traveling the road. The maximum static strains recorded in the steel deck were 139 micro-strain. The minimum strains were -128 micro-strain. The largest range of strain was 187 micro-strain. The experimental strains were compared to analytical strains. Analytical strains were calculated with two programs: FINPLA2 and ABAQUS. Tensile strains and the range of strain in the wearing surface varies with modulus and thickness.

This report presents charts for selecting the thickness of a wearing surface. The charts show strain vs modulus and thickness. These charts were developed to give engineers and suppliers a method for selecting alternate surfaces. Similar charts were developed to determine thermal strains and stresses in the wearing surfaces. The study showed that tensile strains and the range of strain for a given wearing surface were low; thermal stresses were high. Cold temperature thermal cracking, abrasion, adhesion to the steel deck, and traction are important parameters for selecting a furture wearing surface. Liveload fatigue in the wearing surface should not be problem for this structure.

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ABSTRACT

A 2900-ft long bridge with six spans was built over the Yukon River in the state of Alaska in the 1970's. The bridge has a 30-ft roadway that carries vehicles, supports the pipeline, is on a 6% grade and is subjected to -50 degree winter temperatures. These conditions make selecting wearing surfaces a difficult decision. The bridge superstructure has an orthotropic steel deck that is overlaid with a temporary timber wearing surface. The timber deck consists of two layers of 3 by 12 boards and is supported by two 61-inch wide by 163-inch deep box girders. This study focused on predicting strain levels of possible alternative wearing surfaces.

Static strains were measured in the steel deck for several trucks traveling the road. The maximum static strains recorded in the steel deck were 139 micro-strain. The minimum strains were -128 micro-strain. The largest range of strain was 187 micro-strain. The experimental strains were compared to analytical strains. Analytical strains were calculated with two programs: FINPLA2 and ABAQUS. Tensile strains and the range of strain in the wearing surface varies with modulus and thickness.

This report presents charts for selecting the thickness of a wearing surface. The charts show strain vs modulus and thickness. These charts were developed to give engineers and suppliers a method for selecting alternate surfaces. Similar charts were developed to determine thermal strains and stresses in the wearing surfaces. The study showed that tensile strains and the range of strain for a given wearing surface were low; thermal stresses were high. Cold temperature thermal cracking, abrasion, adhesion to the steel deck, and traction are important parameters for selecting a furture wearing surface. Liveload fatigue in the wearing surface should not be problem for this structure.

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EXECUTIVE SUMMARY

Overview

It was the purpose of this project to review the literature on available wearing surface alternatives for orthotropic steel deck bridges and to measure strains in the orthotropic steel deck due to loaded trucks. The procedure proposed to perform the research for the AKDOT&PF project entitled "Yukon River Bridge, Deck Strains and Surfacing Alternatives" included the following tasks:

Task 1:	Proposal Completion and Approval
Task 2:	Literature Summary
Task 3:	Field Testing Summary
Task 4:	Semi-Annual Report
Task 5:	Field Instrumentation Completion Report
Task 6:	Final Draft Report
Task 7:	Published Report Distribution
Task 8:	Presentations on Study Results

Summary of Findings

The authors conducted a literature review of wearing surfaces for orthortropic steel deck bridges and a survey of DOT's that asked for information on wearing surface experiences. While on a field trip to the Yukon River Bridge during the summer of 1993, researchers took static strain measurements of the steel deck for several trucks at different positions. Later a comparative analysis of alternative wearing surfaces was conducted. The research methods used in this report suggest the following wearing surfaces are worthy of testing in Phase 2:

- Cobra X Grade crossing surface modules over a timber deck and membrane
- Polymer concrete bonded to the steel with an epoxy tack coat
- Duraphalt (cracked asphalt cement composed of various resins, polymers and special fillers)
- Epoxy asphalt over a coal tar pitch epoxy
- Gruss asphalt, mastic asphalt
- Polyurethane elastomers impregnated with stone chips
- Epoxy asphalt over an epoxy binder coat
- Transpo T-48 epoxy concrte
- Polymer modified asphalt concrete

The Yukon River Bridge has an orthotropic steel deck that is overlaid with a five-inch two-layer temporary timber deck wearing surface. For this condition, strains were recorded in the steel orthotropic deck of the Yukon River Bridge for loaded trucks during the month of September, 1993. Static strains at 13 deck locations were measured for trucks stopped at various positions on the bridge. Strains were also measured on the deck at midspan between floor beams under moving truck traffic.

Static Tests. Maximum strains recorded in the steel deck were 139.1 micro-strain. Minimum strains were -127.7 micro-strain, and the largest range in strain was 186.9 micro-strain. These live load flexural strains were low, indicating that conventional wearing surfaces may be a viable alternative for the structure.

Dynamic Strains. Strains were recorded for the steel deck at midspan between floor beams for normal truck traffic. The largest strains recorded for these tests were 46.4 micro-strain and -72.8 micro-strain. The maximum range of strain was 119.2 micro-strain. Comparable static strains for this same gauge location, Gauge 4, were a maximum of 76.4 micro-strain, a minimum of -127.7 micro-strain, and a strain range of 186.9 micro-strain.

General Comments. The magnitude of the measured live load flexural strains in the deck was very low. This low magnitude suggests that alternative wearing surfacing materials may be suitable for this bridge. Based on the results of the analysis and experimental data, two factors should be studied in the laboratory when selecting a wearing surface for this structure: the material selected must provide sufficient traction for the 6% grade found on the Yukon River Bridge deck, a shear strain consideration, and sufficient bond strength must be available between the steel deck and the surfacing material to carry interface shear strains under extreme temperature conditions. The analysis indicates that thermal cracking may be a significant problem; materials should be tested to study this condition.

A method is presented for selecting a wearing surface for the Yukon River Bridge to resist live load flexeral strain and thermal cracking.

INTRODUCTION

The goal of this study was to provide results that can lead to the selection of possible surfacing alternatives over the steel deck at the Yukon River Bridge. The project was conducted with the idea that this would be the first of three phases. The objectives for each phase are presented below:

1. **Phase 1 (this study).** Both thermal and truck induced strains were to be measured in the deck of the existing Yukon River Bridge. The experimental results were to be correlated with a computer model to provide a mechanism to predict interface strains for surfacing alternatives. The performance of surface alternatives used on similar bridges such as the "Golden Gate Bridge", the "Throgs Neck Approach Viaducts", and the "Benjamin Franklin Bridge" (Wolchuk, 1987) would be studied through a literature review and questionnaire to SDOT bridge engineers. The results from the literature review were to provide insight into promising surface alternatives for orthotropic of bridge decks.

2. **Phase 2 (next study).** Using the information from Phase 1, surfacing alternatives were to be selected for study. Prior to study, analytical interface strains predicted for the loads and conditions at the Yukon River Bridge would be calculated using a computer model. Laboratory material tests would then be conducted to simulate field strains and loading conditions. It would be the objective of phase 2 to evaluate, through laboratory studies, the performance of possible surfacing alternatives for the bridge deck.

3. **Phase 3 (third study, field application).** Instrument and monitor the performance of field sections at the Yukon River Bridge. The objective of this phase is to provide bridge engineers with performance data for experimental surfacing alternatives. The results from this phase should provide bridge engineers with data that may improve economical long term decisions.

History

Orthotropic Steel Deck Bridges (OSDB) employ stiffened steel plates to carry vehicle loads directly to main supporting members. This design is in contrast to conventional systems which use concrete or timber decks supported by beams and main supporting members.

Orthotropic decks were first introduced in the early 1930's for use with moveable bridges (Heins and Firmage, 1979). These decks have steel plates supported by longitudinal, rolled stringers. The full advantages inherent in the system were not realized until after World War II when, due to a shortage of structural steel, the Germans started experimenting with non-traditional bridge structures. In the 1950's, improved analytical methods and new construction techniques lead to the acceptance of this system.

Advantages and Disadvantages

Orthotropic steel deck systems in long-span structures have the advantage of considerably reduced dead weight when compared with composite girder structures. Disadvantages of orthotropic steel deck structures include: 1) corrosion protection of the steel deck, which is critical to structure longevity, may be adversely affected by de-icing agents; 2) the steel deck is susceptible to abrasion, leading to a loss of strength and/or structural integrity; 3) thin wearing surfaces, intended to improve road surface traction and protect the steel deck, have not performed well; and 4) wearing surface low life-cycles (due to a number of influences) have been a serious concern.

Problem

Orthotropic decks fall into two categories: a) open cellular and b) closed cellular systems. The Yukon River Bridge is a closed cellular system. Therefore the purpose of this study is to identify alternative cost-effective wearing surfaces for closed cellular decks that will do the following:

- survive environmental extremes associated with Alaskan highways,
- minimize abrasion of the steel deck,
- maximize vehicular traction,
- protect the steel deck from agents that induce corrosion,
- minimize maintenance costs,
- provide sufficient bond/adhesion to allow use on steep grades, and
- maximize life cycle and produce an optimal benefit/cost ratio.

Wearing surfaces (systems) are generally categorized as either thin or thick. Thick systems are further classified as rigid or flexible. Some thick systems are modular, others are installed as continuous systems.

Thin systems (less than one inch in thickness) are obtained by applying a thin layer of adhesive/cement matrix directly to the steel deck. Most of these systems employ some version of a fine grit embedded in the matrix to achieve adequate traction. Troitsky (1987) reports that a desirable life for a thin wearing surface is at least five years. The short design life of thin wearing surfaces appears to be due to high wear rates experienced by these materials. Additionally, thin systems are not effective in masking the inherent waviness of the steel deck, making a planar surface difficult to achieve. Application of a thin system requires extensive surface preparation and rigorous adherence to quality control through on-site inspection. New materials, currently under development, may lead to more successful thin surface alternatives.

Thick systems include asphaltic concrete overlays, Portland Cement Concrete (PCC) deck systems, timber, and timber/composite surfaces. PCC systems may consist of precast modularized elements composed of normal or lightweight concretes. Timber systems include solid sawn, glued-laminated or nail-laminated members, often in conjunction with other materials to form a composite system.

A further category "other" is used to include systems that can not easily be classified as either thick or thin systems. These include proprietary systems and high-tech composites currently under development. The literature shows that a suitable wearing surface must have the following characteristics (Troitsky 1987):

- Sufficient ductility to accommodate, without cracking or delamination, any expansion or contraction of the steel plate.
- Sufficient fatigue strength to withstand flexural cracking due to deck plate deflections.
- Sufficient durability to resist rutting, shoving and wearing.
- Imperviousness to water, motor fuels and oils.
- Sufficient resistance to deterioration from de-icing chemicals and petroleum distillates.

Experiences with orthotropic deck wearing surfaces by other state agencies may be of limited value for the following reasons:

- The Yukon River orthotropic steel deck bridge may have steeper grades (6+ %) than is common in other areas.
- Conditions imposed by extreme temperatures, heavy truck loads, and low volumes of traffic may not allow extrapolation of results obtained by others.

- The deck/bridge flexibilities for the Yukon River Bridge may be different from those found for other areas.

- The amount of snow plowing and/or the use of deicing chemicals may be different for the Yukon River Bridge.

The selection of a wearing surface is a complicated process that should consider both structural and traffic performance. These are determined by the selection of wearing surface materials and a fastening system and their compatibility with the orthotropic deck. All of these elements comprise a system. Decision criteria must reflect the entire system and not concentrate on individual elements. The system is affected by a large number of factors which interact. Reaching the optimal solution requires considering all the factors affecting the entire system. Based on the above criteria, the following questions arise and should be answered to determine what kind of orthotropic steel deck bridge wearing surfaces will be suitable on the Yukon River bridge.

- What affect will sub-zero temperature extremes have on the fatigue performance of the bonding systems (adhesives/mechanical fasteners) between the steel plate and the wearing surface?

- What relationship exists between percent grade and shear force in the wearing course?

- How does the life of a wearing surface decrease with the use of snow plows and tire chains?

- What produces most of the abrasion of a wearing surface, and how can this be reduced?

- How can traction be improved and maintained over long periods of time?

- How does the flexibility of an orthotropic steel deck influence the long term performance of the wearing surface?

- What are the criteria for selecting a thin wearing surface versus a thick wearing surface?

- Are unproven wearing surfaces a reasonable alternative for consideration?

- What kind of system will provide sufficient protection to the orthotropic steel deck and maximize structural integrity?

This study (Phase 1) was focused on determining the expected magnitude of live load strain for different types of wearing surfaces. A goal of this study was to learn from other agencies' experiences and from literature on similar types of bridge decks.

Methodology

The reader is advised that the Yukon River Bridge Research activities involved several steps. Each step is listed with the work status, which should provide an understanding of research progress. This study was conducted with the following approach:

1. Review literature:
   a) Search for past experiences with alternative wearing surfaces for orthotropic steel deck bridges,

   b) Evaluate the available analytical methods for calculating strains in orthotropic steel decks;

2. Conduct a two-part national survey of DOT's:
   a) Question to assimilate DOT experiences with wearing surfaces on orthotropic steel deck bridges,

   b) Question to identify computer programs used by DOT's to analyze and design orthotropic steel deck bridges;

3. Obtain computer programs to analyze steel deck strains;

4. Install computer programs & test for known solutions;

5. Calculate preliminary deck strains to locate strain gauges;

6. Conduct a site visit to organize the field test methodology;

7. Order strain gauges and instrumentation installation materials;

8. Conduct laboratory tests on instrumentation equipment;

9. Instrument the Yukon River Bridge and measure deck strains:
   a) Install strain gauges on the underside of the steel deck,

   b) Conduct both static and dynamic strains for truck traffic;

10. Obtain truck weights from the Fox scale station;

11. Calculate Yukon River Bridge deck strains for the truck loads used for testing and compare the results' analytical strains with the experimental strains;

12. Calculate stiffness ratio of wearing surface divided by deck;

13. Summarize final results.

LITERATURE REVIEW

Orthotropic Bridge Deck Wearing Surfaces

Wearing surfaces on steel plate decks should be lightweight, have sufficient thickness to cover deck irregularities, provide skid resistance, have stability and durability over the expected temperature range, provide corrosion protection for the deck, maintain adequate bond with the steel, be resistant to rutting and fatigue, and have a long life (Fondriest, 1968a, 1968b, 1969; Stahl, 1989; Woehlk, 1985; Labek, 1982; IABSE, 1968; Rooke, 1968; Davis, 1969; Moore, 1972; Victor, 1978; Gaddis, 1989, 1990; Risch, 1971; Patterson, 1971). Wearing surfaces for orthotropic steel deck bridges are usually identified as thin or thick; a thin wearing surface is less than 1" thick. In the past, the performance of thin wearing surfaces has been less than satisfactory ($\leq$ 5 year life).

The Yukon River Bridge has three unusual features: a 6% grade, extreme temperatures during the winter months, and a low volume of heavy loads. <u>No research data has been found in the literature to suggest wearing surfaces used on similar bridges will perform satisfactorily on the Yukon River Bridge.</u> Factors that may significantly influence wearing surface life are surface traction, cold weather fatigue, permeability, ductility, abrasion resistance, and shear resistance of the bonding material at the steel interface (Troitsky, 1987).

Examples of wearing surfaces for selected orthotropic steel deck bridges in North America are shown in Table 1. Gaddis and Clark (1990) provided a listing of about 20 orthotropic steel bridges that have been built in the United States, see Table 2. Research by others suggests that asphalt with additives may provide cost effective solutions for orthotropic steel deck bridges. For example, a wearing surface should provide protection to the deck, be lightweight, be durable, and be fatigue resistant. Others have found that the mastic or bonding agent between the steel and the wearing surface is extremely important.

Table 1. Existing Wearing Surfaces on Orthotropic Deck Bridges

Bridge Name	Wearing Surface	Bonding Material
George Washington	Bituminous Asphalt, 1 1/2"	
Golden Gate	Epoxy Asphalt, 2"	Epoxy Seal Coat
Throgs Neck	Bituminous Asphalt, 1 1/2"	Epoxy Seal Coat
Benjamin Franklin	Bituminous Asphalt, 1 1/4"	Epoxy Asphalt, 1 1/4"
Yukon River Bridge	Two part timber deck and 2 experimental test sections(1)	

(1) One test section consisted of treated timber stringers and runners; the other was composed of 18"x52" Cobra X Grade Crossing Surface Modules attached with 7/8" diameter by 4" long lag screws to a 3x12 timber plank underlayment.

Table 2. Orthotropic Bridges in the United States (after Gaddis & Clark, 1990)

Bridge Name & Location	Year Completed	Length(ft)
Humphrey's Creek Bridge, Sparrows Pt., Maryland	1965	112
Dublin Bridge, Hwy 680, Livermore, California	1965	320
San-Mateo-Hayward Bridge, Hayward, California	1967	5,542 of Orthotropic Spans
Poplar Street Bridge, St. Louis, Missouri	1968	2,165
Creyt's Rd. Bridge, I-496 near Lansing, Michigan	1968	192
San Diego/Coronado Bridge California	1969	1,870 of Orthotropic Spans
Queesnway Bridge, Long Beach, California	1969	1,200

Table 2. (Cont.) Orthotropic Bridges in the U.S. (after Gaddis & Clark, 1990)

Bridge Name & Location	Year Completed	Length(ft)
Fremont Bridge, Portland, Oregon	1973	2,159
Yukon River Bridge, Alaska	1975	2,300
George Washington Bridge*, New York	1978	4,760
Throgs Neck Bridge*, New York	1984	13,410
Golden Gate Bridge*, San Francisco	1985	8,981
Benjamin Franklin Bridge*, Philadelphia, Pennsylvania	1987	7,412

*Bridge Decks were replaced with orthotropic steel plate decks

Cullimore, Fleet, and Smith (1984) at the University of Bristol conducted a significant number of tests to evaluate the performance of asphalt wearing surface mix designs and bonding agents on orthotropic steel decks. Findings show that deck preparation, the proper bonding agent, temperature exposures, and deck flexibility are extremely important. Researchers at the University of Bristol found that wearing surface deterioration due to fatigue cracking and rutting is extremely important and can be significantly influenced by the bonding medium between the steel plate and the bituminous surface. The bonding layer must sustain high shear forces at the steel/asphalt interface. The data from the University of Bristol suggest that an epoxy asphalt placed on a membrane composed of coal tar pitch epoxy compound was the most promising surfacing material.

Stahl (1989) reported that extensive testing and field experience has shown that epoxy asphalt applied over an epoxy binder coat to blast-cleaned steel or blast-cleaned and inorganic zinc-coated steel provides a stable wearing surface for long-time service. Satisfactory riding qualities with reduced durability in comparison to epoxy asphalt can be obtained with bituminous asphalt pavement applied over an epoxy binder course into which sand or fine stone particles have been cast to provide an adhesive bond.

Fondriest, (1968a) reported the results of studies of thin wearing surfaces for Orthotropic Bridges. A thin wearing surface is desirable, provided it performs well with time. Several thermosetting materials were selected for investigation. These were coal tar epoxy, oil extended epoxy, polyester, polyamide modified epoxy, polyurethane, and epoxy asphalt. Results of laboratory studies were compared with the field performance of four bridges with similar materials. Comparisons show that if precautions are taken during installation, epoxy mortars could be a suitable wearing surface material. Epoxy-grit mixtures with thicknesses less than 3/8" are questionable.

Fondriest (1968b) reported that nine U.S. orthotropic bridges paved with thick wearing surfaces were studied. Performance after over three years in-service were mixed. The primary problems are fatigue cracking of asphalt concrete and low bond strength between the pavement and the steel deck. The report's laboratory and field studies indicate that epoxy asphalt may provide an excellent paving material. A summary of the materials studied is shown in Tables 3 and 4.

Table 3. Details of Thick Wearing Surfaces on Orthotropic Bridges (after Fondriest, 1968b)

Bridge	Date in Service	Surfacing Details				Remarks
		Prime Coat	Bond Coat	Leveling Course	Wearing Surface	
Port Mann	June, 1964	Red Lead Epoxy	Coal Tar Epoxy	3/4″ SA[a]	1-1/4″ AC[b]	
Humphreys Creek	July, 1964	None	Coal Tar Epoxy	1″ AC	1″ AC	east half
	July, 1964	None	Coal Tar Epoxy	1″ AC-L[c]	1″ AC-L	west half
Ulatis Creek	Sept., 1965	Inorganic Zinc	None	None	1-1/4″ AC	1/5 of section
	Sept., 1965	Inorganic Zinc	None	None	1-1/4″ EAC[d]	1/5 of section
Concordia	Aug., 1965	None	Coal Tar Epoxy	None	2″ AC	
Dublin	Dec., 1965	Zinc Metallizing	Coal Tar Epoxy	None	2″ AC	1/4 of section
	Dec., 1965	None	Coal Tar Epoxy	None	2″ AC	1/4 of section
Battle Creek	May, 1967	None	Coal Tar Epoxy	None	1-3/4″ AC	east half
San Mateo	Nov., 1967	Inorganic Zinc	None	3/4″ EAC	3/4″ EAC	
Poplar Street	Nov., 1967	Inorganic Zinc	Coal Tar Epoxy	1-1/4″ AC-L	1-1/4″ AC-L	
Longs Creek	Dec., 1967	Inorganic Zinc	FG[e]	None	1-1/2″ AC	

[a] SA = sand or sheet asphalt
[b] AC = asphalt concrete
[c] AC-L = rubber latex modified asphalt concrete
[d] EAC = epoxy asphalt concrete
[e] FG = fiber glass impregnated with asphalt emulsion and sealed with mastic asphalt

Table 4. Systems for Anchoring Bituminous Wearing Surfaces to Steel Deck Bridges (after Fondriest 1968b)

Bridge	Primer lbs/yd²	Chips lbs/yd²	Gradation - percent passing										Leveling Course
			1/2″	3/8″	3/4″	1/4″	#4	#6	#10	#16	#20	#30	
Troy[1]	1.0	15-18										100	AC & SA
Port Mann	2.8	7.5					100						SA
Humphreys Creek	1.85	4						100			10		AC & AC-L
Ulatis Creek	None	None											AC
Concordia	2.5	2.5					100	50					AC
Dublin	6.6[2]	7.5[3]	100										AC
Battle Creek	2.5[3]	10[3]				100							AC
Poplar Street	1.0	5-8			100		90		10	5			AC-L

Notes: AC = asphalt concrete; SA = sand or sheet asphalt; AC-L = rubber latex modified concrete

[1] Small test bridge that was used to evaluate the wearing surface on the Poplar Street Bridge

[2] Applied in two equal coats before and after chips were applied

[3] Estimated

Lebek (1982) reports that Gussasphalt is a favored surfacing material in Germany and is similar to a mastic asphalt that is used in the United Kingdom. Patterson (1971) reported performance for epoxy mortar wearing surfaces for the Crietz Road Bridge in Lansing Michigan. The purpose of the study was to evaluate if an epoxy wearing surface was practical for the Michigan climate. On the south half of the bridge, an oil-modified epoxy was used for the binder (Guardkote 250), and on the north half, the epoxy binder was composed of two sources (a modified resin and a modified polymide curing agent). The surface was a minimum of 5/8" thick. Skid resistance values for these materials are shown in Table 5.

Table 5. Deck Surface Skid Test Summary; Wet Sliding Tests at 40 mph (after Patterson, 1971)

Mortar Type and Location	Coefficient of Friction[a] and Dates Tested			
	Dec 2, 1969 (initial)	May 4, 1970	Oct. 14, 1970	Jun 3, 1971
North Half (E15-V140)				
Northbound Lane	0.67	0.52	0.57	0.41
Southbound Lane	0.66	0.53	0.55	0.38
Average	0.67	0.53	0.56	0.40
South Half (Guardkote 250)				
Northbound Lane	0.75	0.48	0.56	0.41
Southbound Lane	0.69	0.46	0.51	0.31
Average	0.72	0.47	0.54	0.36

[a] Each test value is the average of 3 individual tests in each direction.

The technical bulletin AE 563, available from Adhesive Engineering Company (1987), provides a performance history of different wearing surfaces for various bridges in North America, see Table 6.

Table 6. Bridge Histories with Concresive Epoxy Asphalt Wearing Surfaces (after Adhesive Engineering Company, 1987)

BRIDGE PROJECT	LOCATION	DATE	THICKNESS	SQUARE FEET	APPROXIMATE TONS	CONDITION AT LAST CHECK
San Mateo Bridge(1) (9/16")	San Mateo, CA	1967	2"	430,000	5600	1986 Excellent
Coronado (3/8")	San Diego, CA	1969	1 5/8"	116,000	1350	1986 OK; manageable cracking after 13 years.
Bay Bridge(2) (PCC)(3)	San Francisco, CA	1969	1/2"	155,000	465	Some ravelling prior to cure; otherwise excellent until entire bridge repaved with denser graded epoxy asphalt.
Queensway (1/2")	Long Beach, CA	1970	2"	96,000	1195	Excellent; some repairs in 1983.
Ross Island and Sellwood (PCC)	Portland, OR Portland, OR	1972 1973	1/2" 7/8"	146,000 47,000	800 220	1977 Excellent wear
MacKay (3/8")	Halifax Canada	1970	2"	128,000	1485	Due to a lack of good compaction, cracks started after 5 years, followed by bond loss; 25% repaved in 1978 with A/C which failed in one year.
MacDonald (PCC)	Halifax Canada	1971	1/2"	119,000	360	Normal wear-concrete underneath deteriorated after 3 yrs. Repaved after 4 yrs.
Fremont (5/8")	Portland, OR	1973	2 1/2"	155,000	2400	Initial ravelling prior to cure, wear not up to expectation because of poor compaction. Overlaid in 1977.
Evergreen Point (PCC)	Seattle, WA	1972	1/2"	270,000	850	1978 excellent normal wear; overlaid in 1982.
Rio Niteroi	Rio de Janeiro Brazil	1973	2 3/8"	220,000	3265	1976 Cracked and lost bond-deck, too flexible at high ambient. Questionable aggregate.

(1) Steel deck plate
(2) Test installation of open graded epoxy asphalt
(3) Portland Cement Concrete
(4) Chip Seal

1987 Table of Bridge Histories with Concresive epoxy Asphalt Wearing Surfaces (cont.)

BRIDGE PROJECT	LOCATION	DATE	THICKNESS	SQUARE FEET	APPROXIMATE TONS	CONDITION AT LAST CHECK
Mercer Bridge (3/8")	Montreal Canada	1974	1 1/2"	21,000	200	Cracks over longitudinal supports.
I-94 Bridges (PCC)	Minneapolis, MN	1973	3/4"	99,000	465	1978 spalls from concrete underneath. Good performance in general.
Lions Gate Bridge (15/32")	Vancouver Canada	1975	1 1/2"	77,000	725	1986 Excellent, normal wear.
San Francisco Oakland Bay Bridge (upper deck) (PCC)	San Francisco, CA	1976	3/4"	1,475,000	6460	1986 Excellent.
San Francisco Oakland Bay Bridge (lower deck) (PCC)	San Francisco, CA	1977	3/4"	1,290,000	5670	1986 Excellent.
Luling Bridge (7/16")	New Orleans, LA	1983	2"	219,000	2700	Minor surface blister repaired. Southern exposure temperature may exceed 180 F. Some distress in wheel track of truck lane.
Ben Franklin Bridge (5/8")	Philadelphia, PA	1986	1 1/4"	632,000	5000	Under construction.
Golden Gate Bridge (5/8")	San Francisco, CA	1986	1 5/8" & 3/8"[4]	576,000	6000 & 600[4]	Just completed.

(1) Steel deck plate
(2) Test installation of open graded epoxy asphalt
(3) Portland Cement Concrete
(4) Chip Seal

A Series of high density polyurethene eslastomers were subjected to cold weather (-70°F) impact testing at the University of Alaska Anchorage (Nottingham, August 1995). Test results showed that these materials remained bonded to high impact strikes at cold temperature; epoxies did not. Based on those test results, the orthotropic steel deck on the Tudor Road Trail Bridge in Anchorage, Alaska was covered with the following system: 1) the steel deck was covered with a prime coat of United Coatings Primer 302 followed by over 40 DFT mils of Elastuff 120. This was immediately followed by a layer of small stone chips. After curing, the bridge deck was paved with conventional asphalt.

YUKON RIVER BRIDGE SITE VISIT

A visit was made to the Yukon River Bridge in early summer to determine the probable location of the instrumentation and assess the complexity of installing sensors on the underneath side of the bridge deck. One of the purposes of this visit was to determine what type of scaffolding would be needed at the time of installation.

During the visit, several observations about the existing bridge deck and the wearing surface performance were noted. First, the nuts holding the expansion joint at the south end of the superstructure were loose and many have fallen off due to vibration. Further, the new timber deck was experiencing rapid deterioration. The new experimental Cobra X crossing modules showed no apparent wear. Although, the time in service at the time of inspection was only a year and no conclusions about long term performance can be drawn, it appears, based on a visual inspection, that the experimental system performance is significantly better than the timber.

RESPONSES TO SURVEY

A two-part survey was sent to bridge designers at 50 state departments of transportation. An example of the survey is presented in the Appendix. The first part of each survey is composed of questions that request information on the department's experience with surface performance of orthotropic steel deck bridges. The second part requested the names of computer programs that the department used in design and analysis of these types of bridges. Responses to this survey are provided in Table 7.

Thirty-eight states responded to the survey, giving a 76% response. Only four of these states have indicated experiences with orthotropic steel deck bridges. A summary of findings for the four states having experience with these types of bridge decks is presented in Table 8 for consideration. The responding states with this type of bridge were Connecticut, Louisina, Michigan, and Missouri.

Table 7. DOT Respondents to Questionnaire

RESPONDENTS	STATE NAME	RESPONDING DATE	EXPERIENCE
1	MICHIGAN	11-04-93	YES
2	VIRGINIA	11-02-93	NO
3	WISCONSIN	11-02-93	NO
4	NEW JERSEY	11-01-93	NO
5	MARYLAND	10-28-93	NO
6	WEST VIRGINIA	10-28-93	NO
7	MARYLAND	10-25-93	NO
8	RHODE ISLAND	10-20-93	NO
9	NEBRASKA	10-20-93	NO
10	OHIO	10-20-93	NO
11	IOWA	10-18-93	NO
12	IDAHO	10-18-93	NO
13	WASHINGTON STATE	10-18-93	NO
14	MISSOURI	10-18-93	YES
15	FLORIDA	10-18-93	NO
16	GEORGIA	10-18-93	NO
17	COLORADO	10-18-93	NO
18	VERMONT	10-18-93	NO
19	TEXAS	10-13-93	NO
20	MAINE	10-13-93	NO
21	NEVADA	10-12-93	NO
22	TENNESSEE	10-12-93	NO
23	ARIZONA	10-12-93	RETURN TO SENDER
24	HAWAII	10-12-93	NO
25	MASSACHUSETTS	10-12-93	NO
26	INDIANA	10-12-93	NO
27	MONTANA	10-12-93	NO
28	WYOMING	10-12-93	NO
29	NEW HAMPSHIRE	10-12-93	NO
30	SOUTH DAKOTA	10-11-93	NO
31	MISSISSIPPI	10-11-93	NO
32	NORTH DAKOTA	10-11-93	NO
33	SOUTH CAROLINA	10-11-93	NO
34	KENTUCKY	10-11-93	NO
35	UTAH	11-30-93	NO
36	MINNESOTA	11-10-93	NO
37	CONNECTICUT	11-08-93	YES
38	LOUISINA	11-10-93	YES

Note: There were 38 respondents or 76%.

Table 8. Summary of Respondent Answers to Survey

Question	Answers			
	Michigan	Missouri	Connecticut	Louisiana
Number of Orthtropic Bridges	a	----	1	2
Long span bridges (>300 *ft*) & number exposed to freezing temperatures	-----	----	none	yes, 1
Type of wearing surface currently used over orthotropic steel bridge decks			Pressure treated timber with top of the steel plate painted	Epoxy concrete and latex modified asphaltic concrete
What type of wearing surface has been found to perform the best (economical with less maintenance)?			----	None
Has your state found an attachment system that has successfully bonded wearing surfaces to steel decks?			no	no
What is the expected life of your best wearing surface over orthotropic steel bridge decks?			12 years	10 years
Have you found a bridge deck wearing surface that will provide traction on steep grades, i.e. about a 6% grade?			no	no
Has your state tested the use of a wearing surfaces on steel orthotropic decks?			no	have a test section
Computer software			in house, stiffness method	none

Additional information provided by the respondents are listed herein for review and consideration.

Michigan

The Michigan respondent provided a report on an experimental project for an orthotropic bridge on Crietz Road (Risch, 1971). The bridge is a two-span continuous structure with spans of 96'-0" and a clear roadway of 32'-6" with two 9" wide brush curbs. The superstructure has a 7/16" stiffened steel plate deck supported by two 54" deep welded plate girders spaced at 24'-0". Twenty-four-inch floor beams transverse to traffic are spaced at 15'-71/2". The deck was surfaced with two experimental epoxy mortar mixtures:

- Guardkote 250, a low strength flexible system that is oil modified epoxy made by Shell Oil of St. Louis, Missouri; and
- A combination of E15 resin and Versamid 140 polyamide curing agent made by General Mills of Kankakee, Illinois. This system had a higher strength, was moderately flexible and had a slower curing binder.

The structure was opened in 1969 and was tested through June 1971. Skid and abrasion tests were performed to evaluate the wearing surface. Deflection and strain measurements of two test vehicles were conducted for this experimental structure. Both static and dynamic values were recorded. Based on the deflection and strain data, researchers determined that design assumptions were conservative. Test vehicle speeds of 15 to 30 mph caused a 15% increase in deflections and strains. The skid resistance for both types of surfaces were low. These surfaces did not perform well to abrasion.

Missouri

The Missouri respondent provided answers to the questions through several research reports (Gopalaratnam, et al, 1989; Gopalaratnam, Baldwin, and Krull, "Application" 1992; Goplaratnam, Baldwin, and Krull, "Performance I," 1992; Goplaratnam, Baldwin, and Krull, "Performance II," 1992).

The Popular Street Bridge is an orthtropic steel plate deck bridge that carries three major highways, I-70, I-64, and I-55, across the Mississippi at St. Louis, Missouri. At the time of testing, the bridge carried approximately 130,000 vehicles and 15,000 large trucks each day. This is a five-span bridge with a length of 2,165 ft. The superstructure consists of two independent bridges with a total width of 113 ft. Each bridge carries four lanes of traffic in one direction and is supported by two box girders. The box girders are 16 ft deep except at the center span and over the two central piers where the depths are 17 ft and 25 ft respectively.

Six wearing surface materials were evaluated for use as a replacement for the wearing surface. Two systems were asphaltic concrete, three were Epoxy concrete, and one was a Methyl Methacrylate concrete. Laboratory tests for these six systems were performed. Flexural fatigue tests were conducted at $0°F$, and cyclic temperatures ranging from $0°F$ to $160°$ F. Surface conditions before and after these tests were evaluated. Besides the laboratory tests, a test section was installed on the bridge and evaluated for a period of about two years.

Field test sections were observed for evidence of rutting, shoving, and other signs of deterioration. Other tests included material tests and six weeks of monitoring deck strains that were compared with the laboratory fatigue data. Although none of the systems exhibited a sufficient margin against cracking, a proprietary epoxy concrete was recommended as the wearing surface replacement. A Transpo T-48 epoxy concrete wearing surface was used as a replacement. The performance of this material is encouraging but is still being studied.

Connecticut

The Connecticut respondent stated that the state has only one structure with an orthotropic steel deck. The structure is a historic covered bridge with two continuous spans of 95 ft and 77 ft. The structure is on a minimal grade, has light traffic volume and the use of tire chains are infrequent. The state of Connecticut used a timber pressure-treated wearing surface over a painted steel deck. No attachment system was recommended. The expected life of the timber deck wearing surface was 12 years.

A computer program used for analysis was developed in house and is based on the stiffness matrix method. It was the opinion of the responder that the software provided reliable results.

Louisiana

During October, 1993, the Louisianna respondent corresponded by letter, providing the completed survey and four research reports on evaluation of alternative wearing surfaces for orthotropic bridge decks (Huval & Associates, 1992). Information provided in the reports and personal communications with H. Ghara of the Louisiana Department of Transportation and D. Huval at Huval and Assoicates are the basis for the following.

A ten-year-old badly detiorated epoxy asphaltic concrete wearing surface on the 208,620 sq ft orthotropic steel bridge was scheduled for replacement. This structure is a 2,700-ft orthotropic bridge with five spans of 250, 495, 1200, 495 and 250 ft, respectively. The 34-ft roadway crosses the Mississippi River near Liula, Louisiana. The traffic volumes are 12,000 to 18,000 vehicles per day; about 10% of the vehicles are trucks. A Polymer Modified Asphaltic Concrete was originally suggested as the replacement material. The choice was based on a study of twenty-five alternative wearing surfaces. The twenty-five wearing surfaces considered in the study are presented herein for review, see Table 9. Seven were considered worthy of investigation, and four were chosen as test

Table 9. Alternate Wearing Surfaces for the Luling Bridge

Wearing surface	History & comments
Concrete Products: 1. High Density Portland Cement Concrete	1. No previous use on steel decks
Epoxy Systems: 2. Transpo T-48 Polymer Concrete Binder System 3. Flexolith Epoxy Binder System 4. UPM Cold Mix 5. Poly-Carb Epoxy 6. Degussa System 330 Methyl Methacrylate Binder System 7. Resurf Broadcast 8. Mark 163 (Flexigrid) 9. Cono/crete 10. Concresive Epoxy Modified Asphaltic Concrete	2. Popular St. Bridge-performed well 3. Popular St. Bridge-unknown performance 4. Popular St. Bridge-performed well 5. Popular St. & Louisiana-mixed results 6. Popular St. Bridge-performance < 2. 7. Used in Alabama-unknown performance 8. Used in Louisiana-performed well 9. Used in Louisiana-poor performance 10. Luling Bridge-poor performance
Coal Tar Modified Epoxy Products: 11. Cicol ET Slurry	11. Florida & Europe-unknown performance
Latex Systems: 12. Latex Modified Asphaltic Concrete 13. Gem-Crete Flex Latex Modified Concrete with Steel Fibers	12. Louisiana-poor performance 13. Previous use unknown
Mastic Systems: 14. Mastic Asphalt Concrete (Gussaphalt) 15. Stone Mastic Asphalt (SMA)	14. Used in France, Germany, & Pennsylvania. Excellent traction. Performs well. 15. Europe, Maryland, & Luling Bridge-performing well except at splices.
Polymer Systems: 16. Styrelf 14-60 17. Polymer Modified Asphaltic Concrete Pavement 18. Polymer Modified Concrete 19. Resurf II Polymer Concrete 20. Hydroplast	16. Popular St., Sunshine, & patch on Luling Bridge-poor performance in Missouri. 17. Sunshine, patch on Luling, Washington state-performing well 18. Louisiana, Texas-performing well 19. Alamaba-used as a patching material, performance unknown 20. France-performance unknown

Table 9. (Cont.) Alternate Wearing Surfaces for the Luling Bridge	
Resin Modified Systems: 21. Resin Modified Pavement (RMP)	21. Used in France-fuel and abrasion resistant but not recommended for steel decks
Rubber Systems: 22. Crumb Rubber Modifier (CRM) 23. Crumb Rubber Modifier 24. Polyster Concrete Overlay 25. Magstone Toppit	22. Used in Arkansas, California, Kansas, Texas-performed well 23. Use on bridge decks unknown 24. Used by Washington state-performance unknown 25. Used in Utah-performance unknown

after Huval & Associates, Analysis and Engineering Study of Wearing Surfaces and personal communication by D. Huvall on Sept. 14, 1995.

sections. A one-year-old test section of the Polymer Modified Asphaltic Concrete appeared to be performing well. The choice was based on performance and economics. The seven alternative wearing surfaces investigated were:

- Transpo T-48 Epoxy Binder System by Transpo Industries;
- Flexolith Epoxy Binder System;
- Gem-Crete Flex Latex Modified Concrete with Steel Fibers;
- Polymer Modified Asphaltic Concrete Pavement;
- Polymer Modified Concrete;
- Resin Modified Pavement (RMP); and
- Crumb Rubber Modifier (CRM)

Four wearing surface systems were selected for study as test sections on the bridge. The four wearing surface systems originally proposed as test sections are 1) Transpo T-48 overlayed with Polymer Modified Asphaltic Concrete; 2) Asphalt-Rubber Stress Absorbing Membrane Interlayer (SAMI) (Crumb Rubber); 3) Polymer Modified Concrete with steel fibers overlayed with Asphaltic Rubber; and 4) Polymer Modified Asphaltic Concrete.

The authors determined through personal communications with the Louisiana Department of Transportation and Huval that deck temperatures became a concern. For example, bridge deck temperatures approached $140°F$ and some believed that temperatures inside the box were higher. Thus, based on additional studies, the material chosen for replacement was not one of the systems that had been chosen as a test section. The material selected as a replacement was chosen for economic reasons and thermal stability at high temperatures. Based on these considerations, a stone mastic asphalt (SMA) was installed as the replacement wearing surface. It was installed between January and the end of April, 1995.

EXPERIMENTAL STEEL DECK STRAINS

Instrumentation.

A preliminary live load analysis was performed on the orthotropic steel deck prior to instrumentating. The analysis was used to determine expected strain levels of the steel deck. Using this information, strain gages were installed on the under side of the deck near the south abutment in the first span, see Figs 1 and 2. One thermistor was installed on the under side of the deck. The purpose was to monitor temperature of the steel deck. The following criteria were used to select the strain gage placement:

- Expected locations for maximum strains; and
- adaptability to scaffold placement.

During the month of September 1993, thirteen 350 Ohm full bridge weldable strain gauges were mounted to the underside of the Yukon River Bridge's orthotropic steel deck. The strain gauge locations and orientations are shown in Figs. 1, 2 and 3. Following installation, both static and dynamic tests were conducted. Marks to locate the front axles were painted on the timber deck for static tests. Fig. 4 shows locations of the paint marks. The cross-sectional view of the location of truck wheels with respect to the steel deck and girder supports are shown in Fig. 5. The intent of the investigators was to static test loaded trucks; these trucks were traveling north from Fairbanks. Weights of the trucks were obtained from the Fox scale computer records. The wheel base for the trucks used for static tests was measured at the bridge site. A summary of the tests are shown in Table 10 and the size and location of the trucks are shown in Table 11.

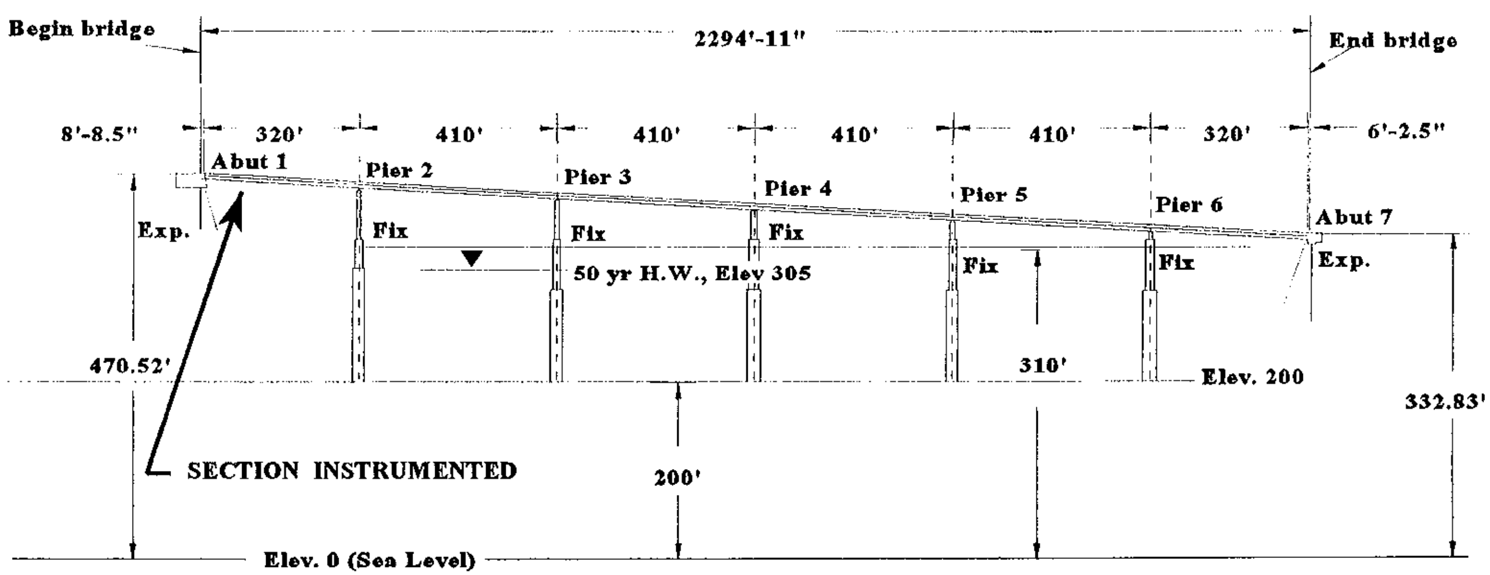

Fig.1. Yukon River Bridge Geometry

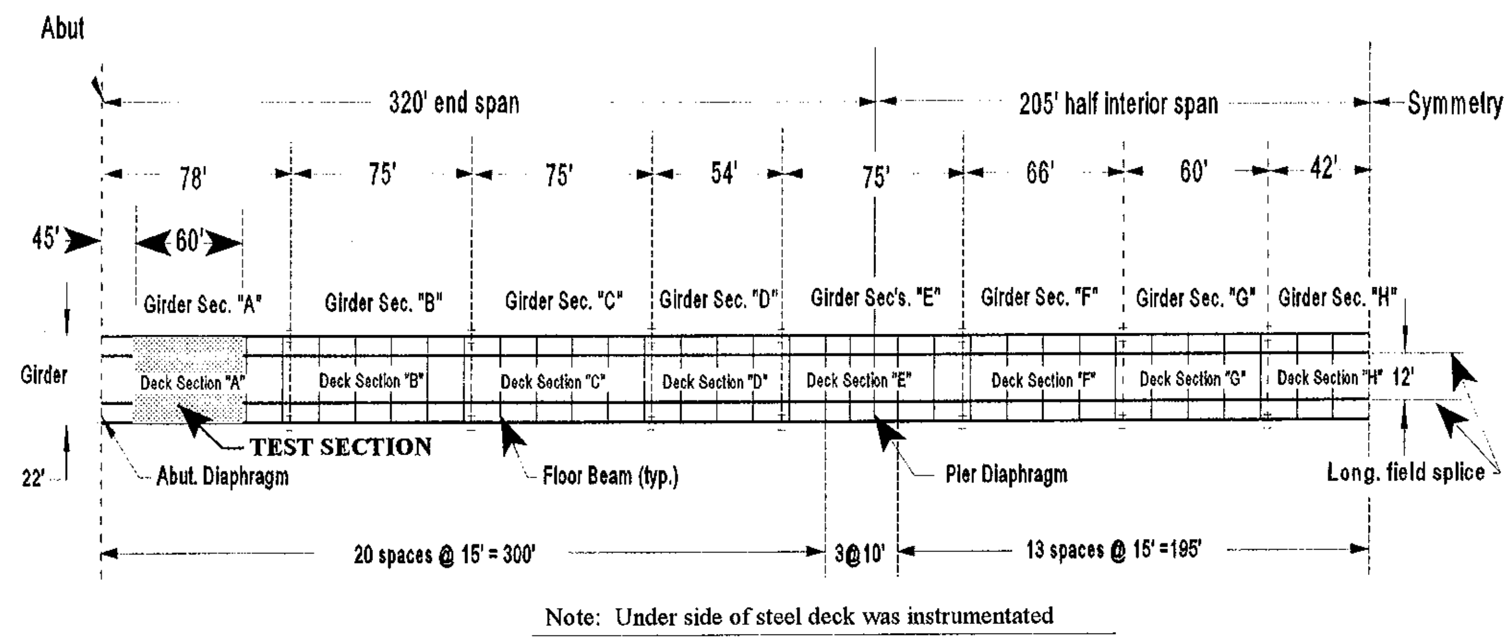

Fig. 2. Orthotropic Bridge Deck Geometry

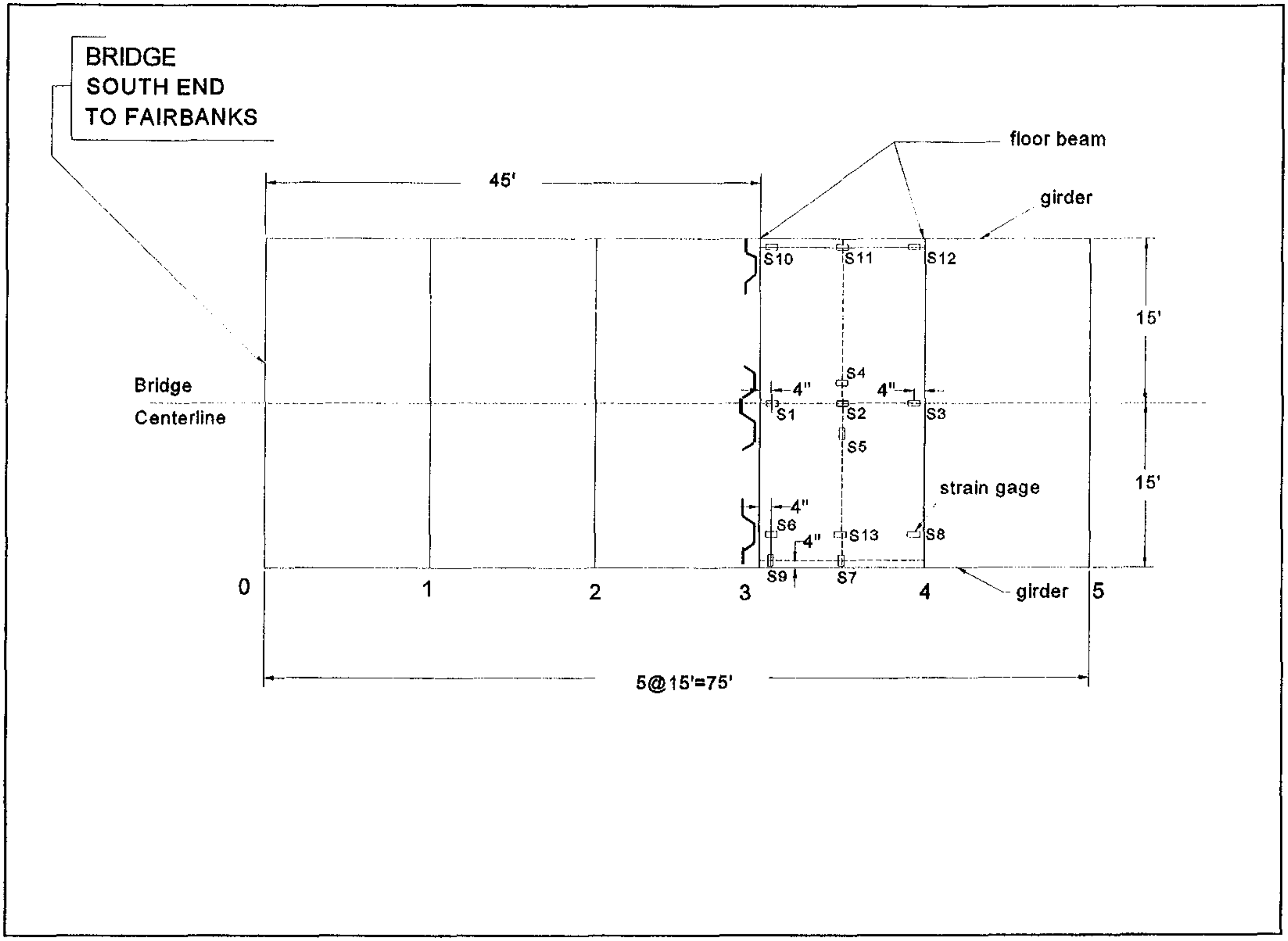

Fig.3. Strain Gauge Positions

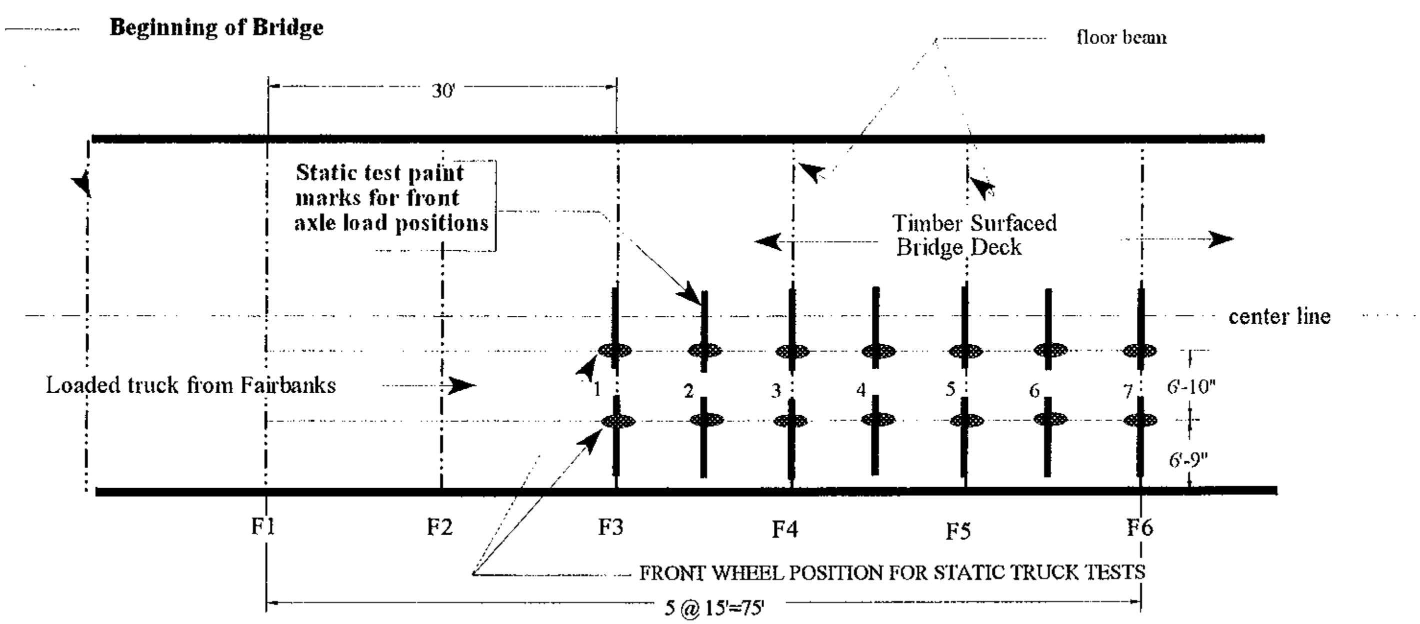

Fig. 4. Test Setup for Static Tests

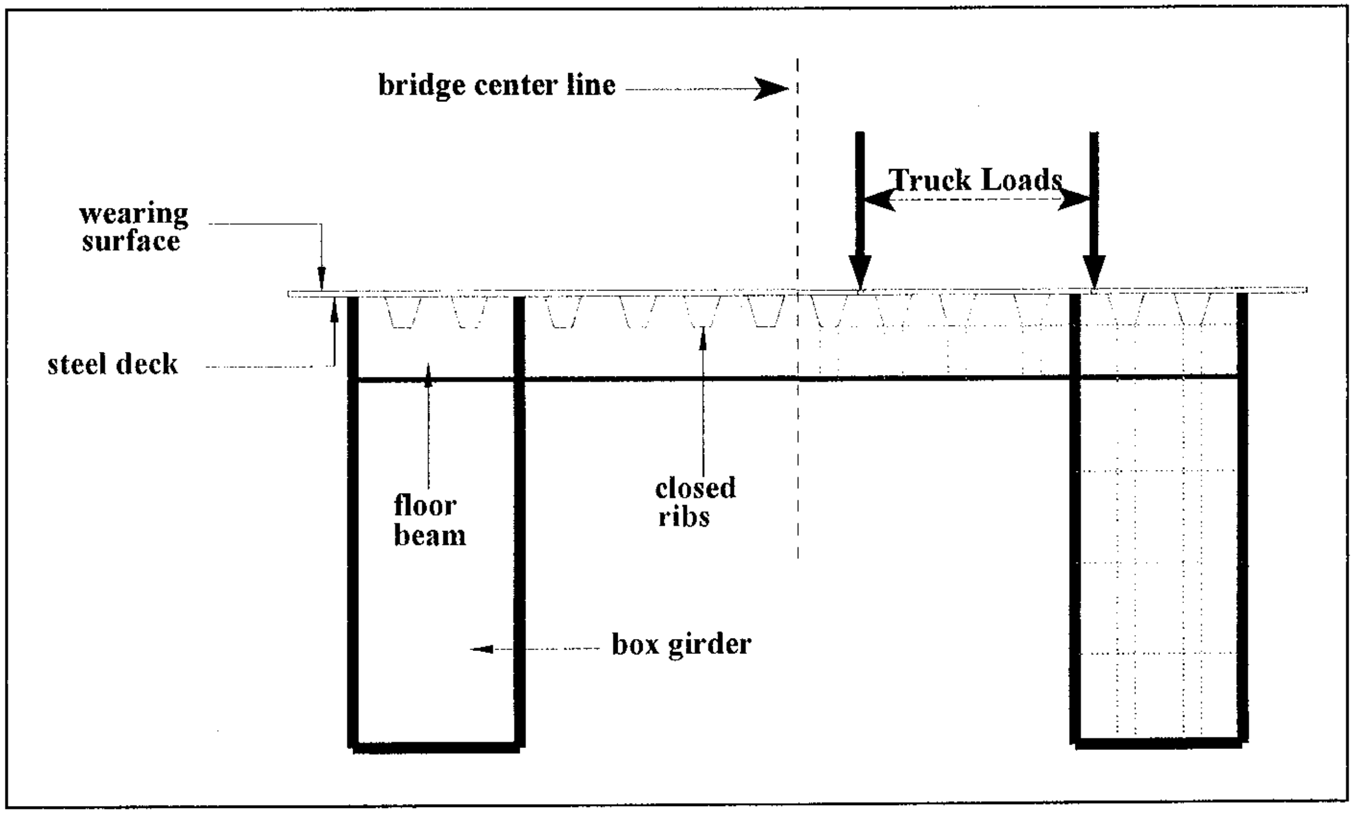

Fig. 5. Superstructure Cross-sectional View

Table 10. Record of Tests at the Yukon River Bridge, 1993.

Test No.	Test Type	Date	Time	Truck Direction	Truck Weight (lbs)[a]		No. of Axles
					Fox Scale	Driver	
Static:[b]							
2	STATIC	9/3/93	15:30:05	Fairbanks-to-North	114,440	115,000	8
3	STATIC	9/3/93	16:31:01	North-to-Fairbanks	92,460	38,000	6
4	STATIC	9/3/93	17:09:01	Fairbanks-to-North			6
5	STATIC	9/3/93	17:25:40	Fairbanks-to-North	88,180	90,000	5
6	STATIC	9/3/93	17:42:17	Fairbanks-to-North		88,000	5
7	STATIC	9/3/93	17:52:01	North-to-Fairbanks		30,000	5
8	STATIC	9/3/93	18:08:37	Fairbanks-to-North		103,000	7
9	STATIC	9/3/93	18:20:00	Fairbanks-to-North		92,000	5
10	STATIC	9/3/93	19:13:19	North-to-Fairbanks			3
Dynamic:[c]							
8	DYNAMIC	9/4/93	13:59:12	Fairbanks-to-North	41,500		3
9	DYNAMIC	9/4/93	14:18:29	Fairbanks-to-North	86,900		3
10	DYNAMIC	9/4/93	14:32:21	Fairbanks-to-North	60,260		3
11	DYNAMIC	9/4/93	14:44:01	North-to-Fairbanks	50,520		3
12	DYNAMIC	9/4/93	14:55:02	North-to-Fairbanks			
13	DYNAMIC	9/4/93	14:58:09	North-to-Fairbanks			
14	DYNAMIC	9/4/93	15:09:35	North-to-Fairbanks			
15	DYNAMIC	9/4/93	15:11:21	North-to-Fairbanks			

[a] Truck weights were obtained through sources; scale weight and driver interviews at the time of testing

[b] Front truck tires were moved over paint marks on the deck and stopped to record strains; 7 different positions

[c] Strains were recorded actual uninterrupted truck traffic; the moving trucks were video taped during testing

Table 11. Truck Measurements for Static Tests

Test #	Truck Width[b] (ft)	Truck Length, Axial Distance From Front Tires (ft)[a]							Side[d] (ft)	Dir.[e]
		2[c]	3	4	5	6	7	8		
1	6'-11"	5'-4"	30'-1"	25'-3"	52'-11"	63'-2"	67'-4"	71'-4"	13'-8"	S-N
2	6'-10"	22'-4"	27'-2"	54'-4"	58'-4"	62'-5"	-----	-----	13'-0"	N-S
3	6'-10"	22'-7"	26'-7"	56'-3"	60'-9"	65'-5"	-----	-----	14'-2"	S-N
4	6'-8"	22'-4"	27'-6"	57'-0"	61'-1"	------	------	-----	15'-7"	S-N
5	6'-8"	21'-3"	26'-3"	54'-5"	64'-6"	------	------	-----	12'-3"	S-N
6	6'-9"	21'-2"	25'-7"	54'-8"	64'-8"	------	------	------	13'-6"	N-S
7	6'-9"	21'-9"	23'-3"	45'-3"	55'-6"	59'-7"	63'-11"	------	18'-8"	S-N
8	6'-10"	21'-5"	26'-4"	57'-1"	61'-4"	------	-------	------	18'-7"	S-N
9	6'-9"	17'-6"	22'-0"	-------	-------	------	-------	------	14'-2"	N-S

[a] Truck length, axle distances measured from center of front tires
[b] Truck width, center-to-center distance between front tires
[c] Number of truck axles
[d] Distance from far front tire to east side of bridge
[e] Direction of travel, e.g. (from Fairbanks was S-N)

Static Tests Results.

Maximum measured strains at the under side of the orthotropic steel deck are presented in Table 12. The deck has a five-inch temporary timber deck wearing surface made of two layers of 3"x12" boards. The timber boards in the top layer are longititutive to the traffic. The maximum strain in the steel deck for the static tests was 139.1 micro-strain; this measurement was found in the

Table 12. Measured Static Strain Extremes (Micro-strain)

	Maximum strains		Minimum strains		Range of Strain	
Gauge	Test	Strain	Test	Strain	Test	Strain
1	9	7.2	7	-0.8	9	7.2
2	5	122.7	10	-12.2	5	109.8
3	2	34.8	5	-14.8	5	35.6
4	9	76.4	7	-127.7	5	186.9
5	5	51.6	8	-29	8	78.7
6	10	5.4	5	-103.2	5	91.8
7	6	55.6	4	-14.2	6	68.2
8	5	139.1	10	-5.4	5	133.8
9	10	8.4	8	-34.4	8	26.4
10	7	70.2	10	-14.1	8	27.1
11	8	44	7	-77.7	7	102.4
12	7	57.3	10	-8.8	7	35.2
13	5	51.6	5	-41.7	5	93.3

longitudinal direction near floor beam 4 near the girder at Gauge 8, see Fig. 3. A minimum steel deck strain of -127.7 micro-strain was in the longitudinal direction, Gauge 4; this gauge is in the middle of the deck between floor beams 4 and 5, see Fig. 3. The largest range of strain was 186.9 micro-strain; this range occurred at Gauge 4. The maximum recorded strain was caused by Test 5.

The test truck was a five-axle 88,180 lb truck. The minimum recorded strain was caused by Test 7; a five-axle truck with an unknown weight. The next smallest strain was caused by Test 5. The largest range of strain was caused by Test 5, an 88,180 lb five-axle truck.

Maximum recorded static strains for each of the 13 gauges are presented in Fig. 7. The minimum recorded static strains for each gauge are shown in Fig. 8. The range of strain for each static test and gauge position is shown in Fig. 9.

Table 8 shows that the Fox scale had only recorded truck axles loads for three of the trucks that were used for the static tests. These were Tests 2, 3, and 5. Test 1 was used to validate the procedure; the data for this test was not considered valid. The Test 2 truck was a 114,440 lb eight axle truck. Test 3 had six axles and weighed 92,460 lbs. The truck used for Test 5 had five axles and weighed 88,180 lbs. The dimensions of each truck are presented truck are presented in Table 11.

Static strains for Test 2 are presented in Table 13. The test weighed 114,440 lb, had eight axles, and was moved to seven different testing positions on the bridge deck, see Tables 10 and 11 and Figs. 2, 3 and 6. The six-axle 92,400 lb truck used in Test 3 was measured at three positions on the bridge deck and the steel deck strains are shown in Table 14. Static strains for Test 4 are shown in Tables 15 and 16. The truck for static Test 5 weighed 88,180 lbs. This was a five-axle truck and was moved to the same seven pre-marked locations on the bridge deck. The measured steel deck strains for this test are given in Table 16. Static strains for the remaining static tests, 6 through 10, are presented in Tables 17 through 21.

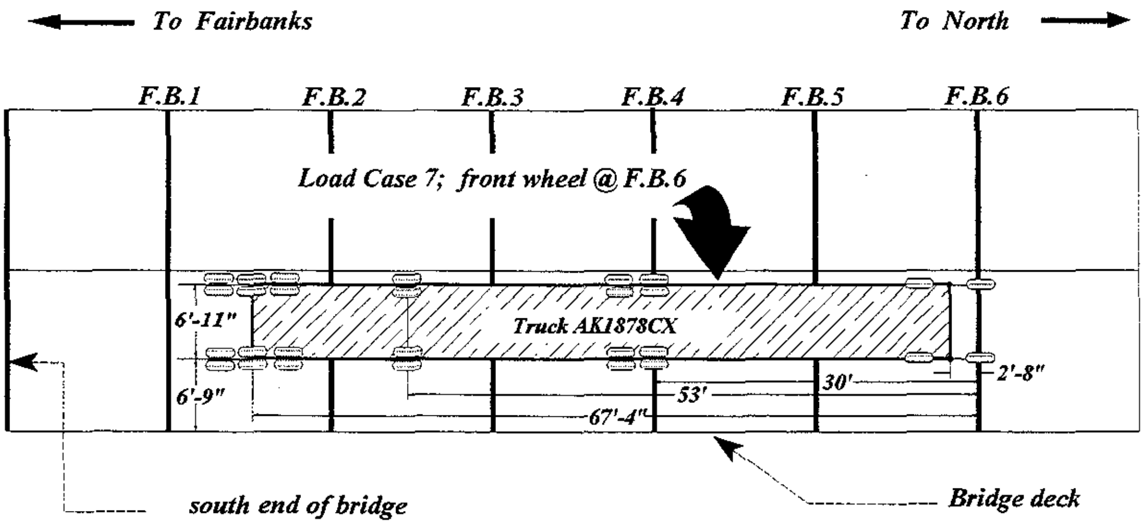

Fig. 6. Bridge Deck Static Testing Plan, Test 2

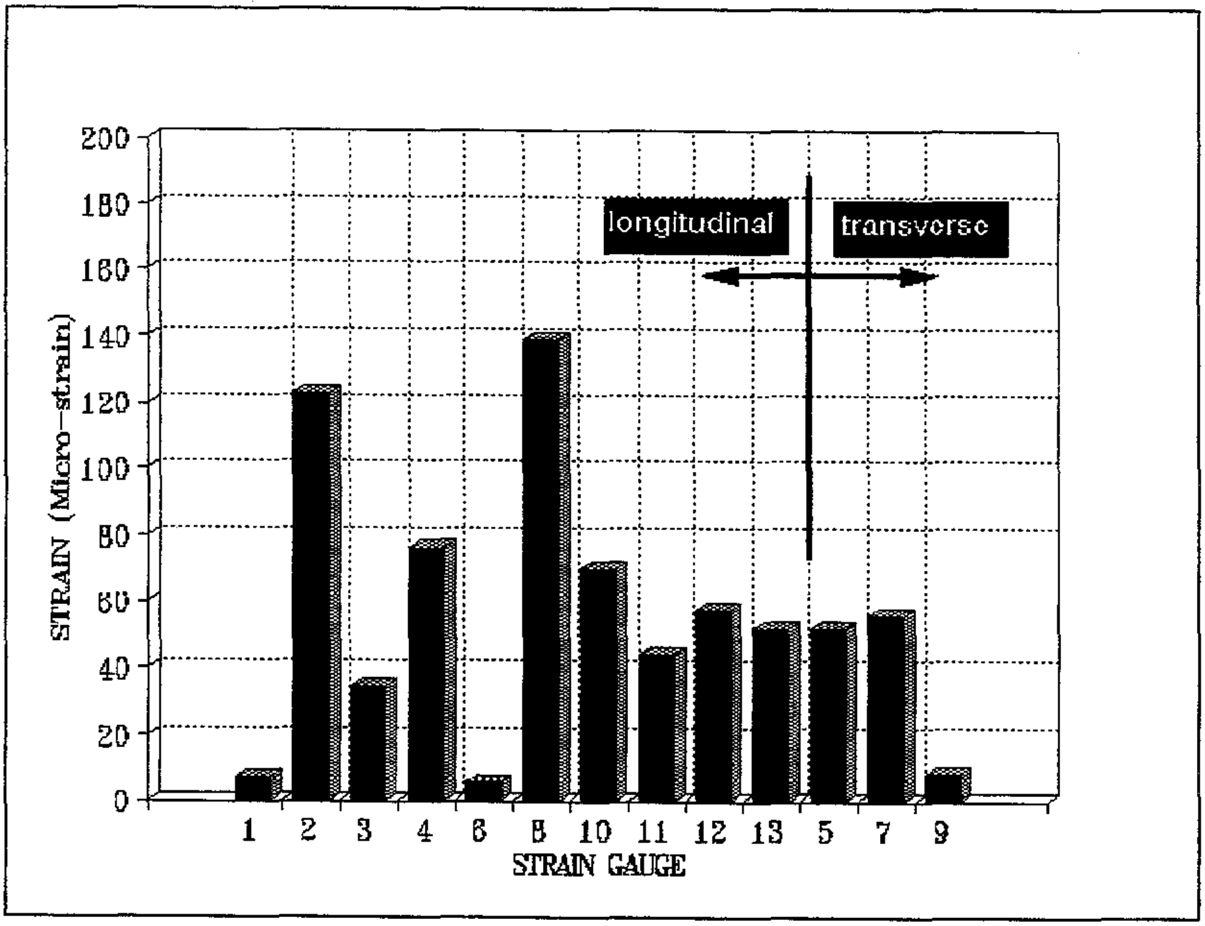

Fig. 7. Maximum Measured Strains at Each Gauge.

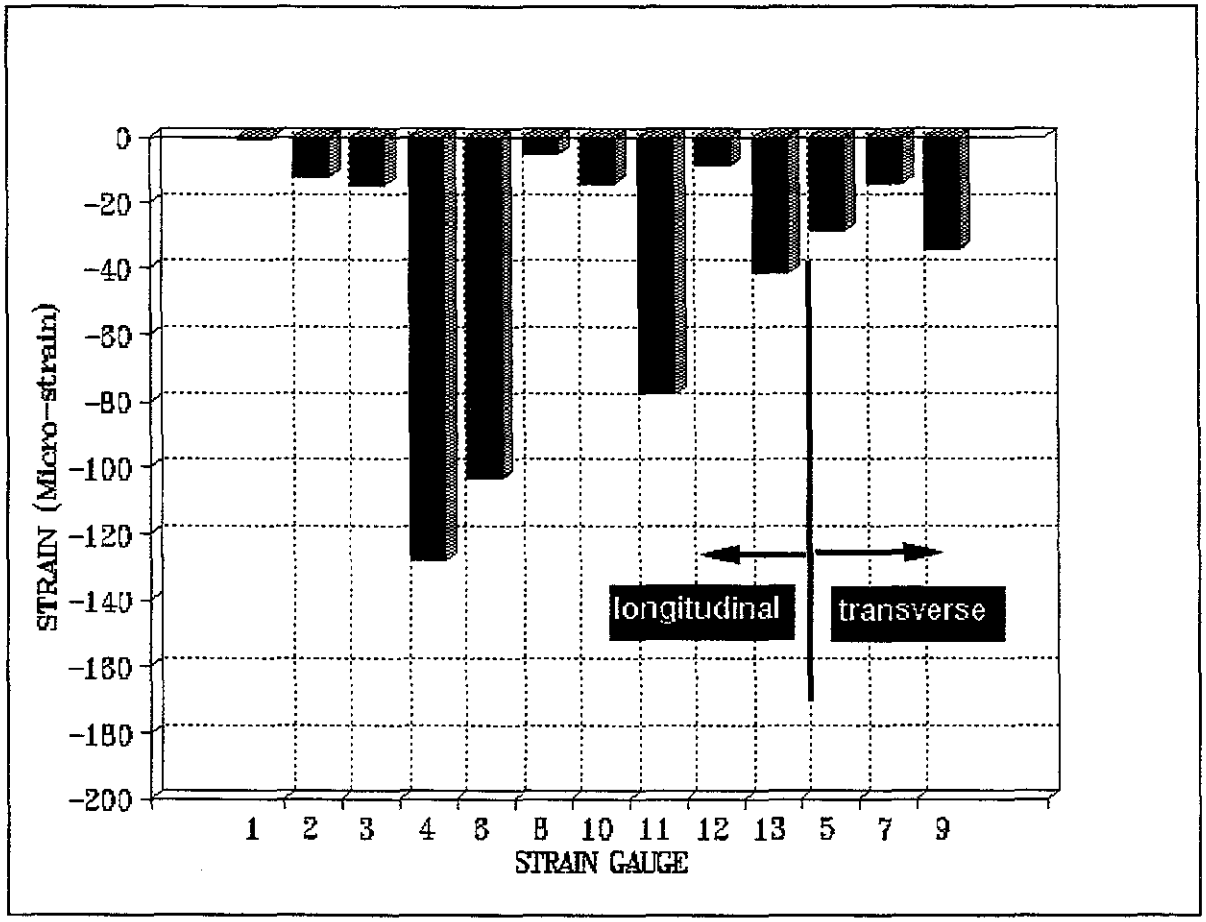

Fig. 8. Minimum Measured Static Strains at Each Gauge

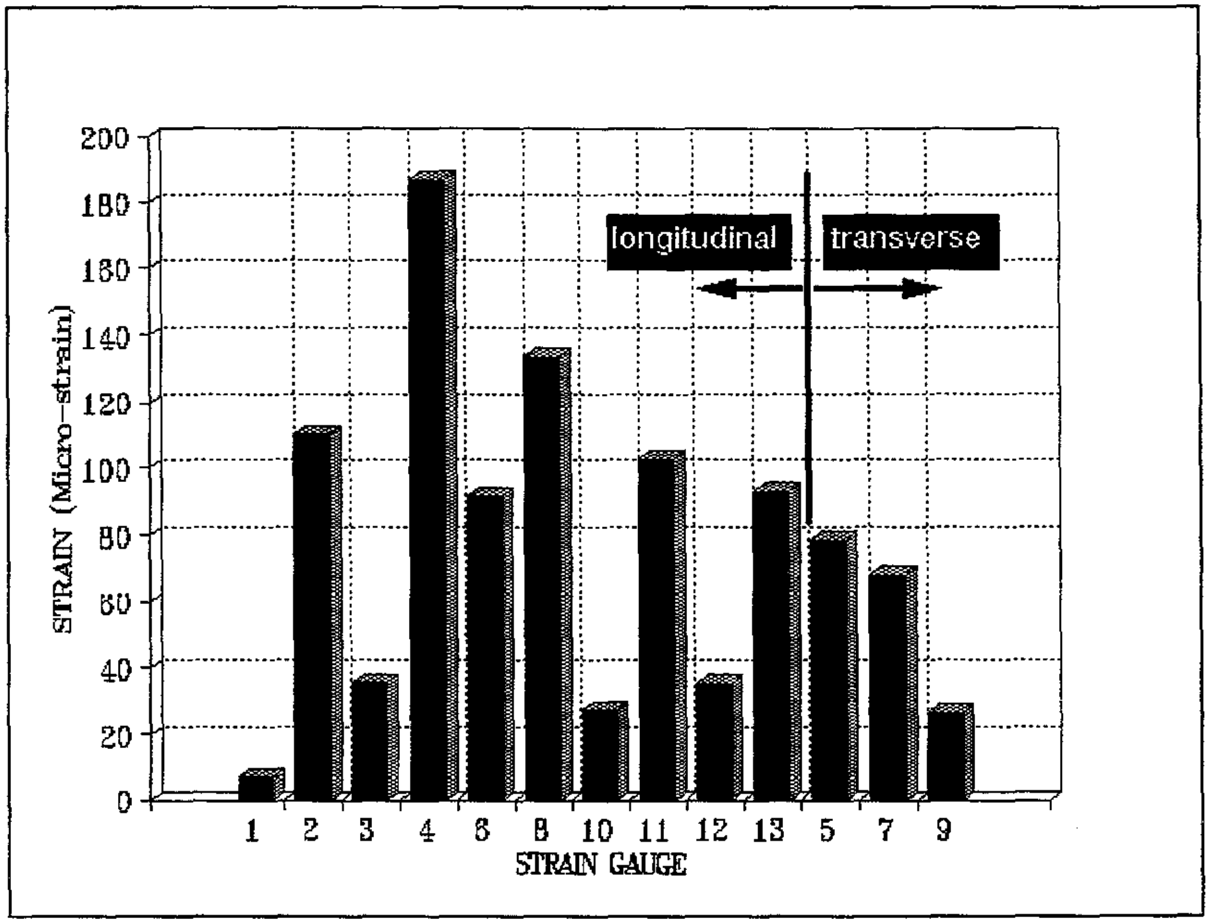

Fig. 9. Range of Measure Static Strain at Each Gauge

Table 13. Measured Strains for Static Test 2.

Gauge Number	Strains (Micro-Strain) at Wheel Positions							Strains (Micro-Strain)		
	1,2	3	4	5	6	7	8	Max	Min	Range
1	3.1	3.1	1.1	2.3	4.6	4.2	1.6	4.6	1.1	3.5
2	12.3	28.7	26	17.6	30.6	47	41.7	47	12.3	34.7
3	11.5	10.7	16.8	19.5	18.3	27.9	34.8	34.8	10.7	24.1
4	16.5	3.1	15.7	37.8	28.3	15.7	32.9	37.8	3.1	34.7
5	-7.6	8.4	-1.9	-19.1	-9.2	5.4	-10.3	8.4	-19.1	27.5
6	-8.8	-15.7	-31.7	-27.5	-24.8	-32.1	-44.3	-8.8	-44.3	35.5
7	-3.4	36.7	0.8	-8.8	-1.9	36.7	-6.1	36.7	-8.8	45.5
8	18	20.3	17.2	24.8	39	38.3	38.4	39	17.2	21.8
9	-11.9	-6.5	-2.7	-9.6	-18.7	-8	-1.8	-1.8	-18.7	16.9
10	4.3	7.6	11.1	11.5	11.1	15	20.7	20.7	4.3	16.4
11	11.1	13.4	16.1	21	24.1	26	29.9	29.9	11.1	18.8
12	8.8	9.6	8.8	13.4	18.3	19.1	19.2	19.2	8.8	10.4
13	12.5	12.6	16.4	27.5	27.1	21	28.7	28.7	12.5	16.2

Table 14. Measured Strains for Static Test 3.

Gauge Number	Strains (Micro-Strain) at Wheel Positions							Strains (Micro-Strain)		
	1	2	3	4	5	6	7	Max	Min	Range
1	-0.4	-0.4	0.8					0.8	-0.4	1.2
2	14.5	21.8	13.4					21.8	13.4	8.4
3	14.1	11.1	10.4					14.1	10.4	3.7
4	13	-17.9	14.5					14.5	-17.9	32.4
5	-5	-3	-3.8					-3	-5	2
6	-8.8	-7.7	-5					-5	-8.8	3.8
7	0.8	2.3	0.4					2.3	0.4	1.9
8	8.4	9.5	10					10	8.4	1.6
9	-10.3	-10.7	-10.7					-10.3	-10.7	0.4
10	17.2	13.4	10.4					17.2	10.4	6.8
11	15.7	13.8	15.3					15.7	13.8	1.9
12	11.1	13.4	16.5					16.5	11.1	5.4
13	9.9	8.4	8.8					9.9	8.4	1.5

Table 15. Measured Strains for Static Test 4.

Gauge Number	Strains (Micro-Strain) at Wheel Positions							Strains (Micro-Strain)		
	1	2	3	4	5	6	7	Max	Min	Range
1	3.1	0.4	-0.3	6.1	3.4	0.7	0.4	6.1	-0.3	6.4
2	11.1	48.2	12.7	14.5	33.4	42	21.44	8.2	11.1	37.1
3	8	3.1	25.7	2.3	2.5	23.7	8.8	25.7	2.3	23.4
4	8.9	-6.9	26.4	34.4	-14.9	-2.7	48.5	48.5	-14.9	63.4
5	-3.8	43.6	-12.6	-17.6	50	20.2	-22.2	50	-22.2	72.2
6	-13	-17.6	-67.3	-44	-30.9	-34.8	-40.5	-13	-67.3	54.3
7	-2.3	12.6	-4.9	-6.1	9.1	1.8	-14.5	12.6	-14.5	27.1
8	13	14.5	6.2	49.7	42	14.9	104.7	104.7	6.2	98.5
9	-0.8	-3.1	-4.2	-7.6	-6.5	-2.7	1.5	1.5	-7.6	9.1
10	4.2	9.2	9.3	5.4	8.4	16.4	21	21	4.2	16.8
11	8.8	11.1	14.6	18	17.9	20.2	26.4	26.4	8.8	17.6
12	7.6	7.6	8.5	14.1	15.3	12.2	15.7	15.7	7.6	8.1
13	9.6	8	23.8	30.2	-2.3	11.4	42.4	42.4	-2.3	44.7

Table 16. Measured Strains for Static Test 5.

Gauge Number	Strains (Micro-Strain) at Wheel Positions							Strains (Micro-Strain)		
	1	2	3	4	5	6	7	Max	Min	Range
1	-1.6	0.3	0.3	1.1	2.3	1.1	1.1	2.3	0.3	2
2	12.6	60.7	12.9	13	122.7	66.7	19.5	122.7	12.9	109.8
3	3	0.7	20.8	0.7	-12.3	6	-14.8	20.8	-14.8	35.6
4	9.9	-62.7	34	47.8	-127.3	-80.2	59.6	59.6	-127.3	186.9
5	-2.7	10.3	-15.3	-20.3	51.6	30.8	-24.1	51.6	-24.1	75.7
6	-11.4	-22.5	-85.9	-103.2	-59.2	-38.2	-32.1	-11.4	-103.2	91.8
7	-0.8	8	-2.7	-7.7	26.7	15.2	-13	26.7	-13	39.7
8	13.3	25.8	5.3	67.6	66.8	27.5	139.1	139.1	5.3	133.8
9	-1.9	-3.8	-8.4	-15.7	-16.6	-9.1	-2.3	-1.9	-16.6	14.7
10	5.3	10.6	12.1	3.8	6.6	19.8	25.2	25.2	3.8	21.4
11	10.2	12.1	18.6	22.5	19.4	20.8	30.9	30.9	12.1	18.8
12	8.7	8.7	9.9	17.5	18.3	11.8	15.3	18.3	8.7	9.6
13	10.3	-13.8	27.8	45.8	-41.7	-10	51.6	51.6	-41.7	93.3

Table 17. Measured Strains for Static Test 6.

Gauge Number	Strains (Micro-Strain) at Wheel Positions							Strains (Micro-Strain)		
	1	2	3	4	5	6	7	Max	Min	Range
1	2.4	0	0.8	5	3.1	1.6	1.6	5	0	5
2	11.2	21.5	14.2	17.2	40.1	36.4	21.8	40.1	11.2	28.9
3	8.5	10.4	16.1	12.6	15.3	26	33	33	8.5	24.5
4	9.6	8.4	22.6	24.8	6.9	17.6	42.1	42.1	6.9	35.2
5	-3	2.7	-10.3	-11.5	8	-1.9	-19.8	8	-19.8	27.8
6	-9.2	-18.4	-26	-14.1	-20.6	-35.6	-36.7	-9.2	-36.7	27.5
7	-3	23.7	-6.5	-3.8	55.6	-5.3	-12.6	55.6	-12.6	68.2
8	13.9	15.7	16.5	26.8	31.7	23.6	28.3	31.7	13.9	17.8
9	-5.8	-1.6	-5	-8	-11.8	0	2.6	2.6	-11.8	14.4
10	4.3	7.4	8.5	6.5	9.6	16.1	17.7	17.7	4.3	13.4
11	8.6	10.8	14.6	17.2	18.7	21.5	25.3	25.3	8.6	16.7
12	6.2	6.9	8.8	12.6	14.5	13.1	16.1	16.1	6.2	9.9
13	8.9	8.8	19.5	22.5	13.8	17.3	33.3	33.3	8.8	24.5

Table 18. Measured Strains for Static Test 7.

Gauge Number	Strains (Micro-Strain) at Wheel Positions							Strains (Micro-Strain)		
	1	2	3	4	5	6	7	Max	Min	Range
1	-0.4	-0.8	1.1					1.1	-0.8	1.9
2	29	61.1	20.6					61.1	20.6	40.5
3	10.3	1.5	11.8					11.8	1.5	10.3
4	1.1	-127.7	16.8					16.8	-127.7	144.5
5	-5.4	1.1	-5					1.1	-5.4	6.5
6	-10.7	-6.9	-1.9					-1.9	-10.7	8.8
7	5.3	9.9	3.8					9.9	3.8	6.1
8	9.1	12.8	14.8					14.8	9.1	5.7
9	-20.6	-24.4	-24.4					-20.6	-24.4	3.8
10	45.4	70.2	58					70.2	45.4	24.8
11	19.1	-77.7	24.7					24.7	-77.7	102.4
12	28.6	57.3	22.1					57.3	22.1	35.2
13	14.5	11	12.2					14.5	11	3.5

Table 19. Measured Strains for Static Test 8.

Gauge Number	Strains (Micro-Strain) at Wheel Positions							Strains (Micro-Strain)		
	1	2	3	4	5	6	7	Max	Min	Range
1	4.6	0.7	0	6.1	1.9	0.4	0	6.1	0	6.1
2	15.3	24	8.8	14.1	77.2	52.4	16.3	77.2	8.8	68.4
3	8.8	11.8	18.7	7.6	1.1	21.4	1.1	21.4	1.1	20.3
4	8.2	5.3	50.8	48.9	-52.8	-16.1	30.3	50.8	-52.8	103.6
5	-2.3	0.4	-18	-16.1	49.7	15.3	-29	49.7	-29	78.7
6	-6.9	-18	-22.5	-2.3	-11.5	-28.3	-31.3	-2.3	-31.3	29
7	0.4	6.1	0.4	4.2	17.9	8.8	-5	17.9	-5	22.9
8	13.8	15.3	12.2	25.6	28.6	13	18.1	28.6	12.2	16.4
9	-9.2	-10.7	-22.5	-34.4	-30.6	-12.2	-8	-8	-34.4	26.4
10	4.6	14.8	28.8	20.3	11.4	28.8	31.7	31.7	4.6	27.1
11	11.8	12.6	27.5	34.4	13.3	22.2	44	44	11.8	32.2
12	11.8	11.8	10.7	25.2	28.3	10.7	27.5	28.3	10.7	17.6
13	9.2	4.2	21.4	22.2	4.9	17.6	32.5	32.5	4.2	28.3

Table 20. Measured Strains for Static Test 9.

Gauge Number	Strains (Micro-Strain) at Wheel Positions							Strains (Micro-Strain)		
	1	2	3	4	5	6	7	Max	Min	Range
1	3.8	0	0.4	7.2	3.4	0.7	5.7	7.2	0	7.2
2	13.8	24.8	10.3	13.7	87.2	51.9	20.2	87.2	10.3	76.9
3	8.4	10.3	19.1	10.3	8.8	28.8	13.7	28.8	8.8	20
4	8.4	-3.8	44.7	50.1	-54.7	-5.5	76.4	76.4	-54.7	131.1
5	-2.3	1.5	-16.1	-15.7	17.9	-0.8	-26.4	17.9	-26.4	44.3
6	-5	-16.4	-25.6	-9.5	-18.3	-34.4	-30.9	-5	-34.4	29.4
7	0.8	6.9	0.4	2.3	16.4	7.2	-3.9	16.4	-3.9	20.3
8	12.6	15.3	11.4	16	33.2	18.7	28	33.2	11.4	21.8
9	-9.2	-10.3	-19.5	-29	-27.9	-18.3	-16	-9.2	-29	19.8
10	3.8	12.9	25.2	15.2	14.9	33.6	27.8	33.6	12.9	20.7
11	11.1	10.7	25.2	30.5	18.1	27.6	42.3	42.3	10.7	31.6
12	10.7	11	11	23.3	25.2	14.9	27.8	27.8	11	16.8
13	8	2.3	21.4	24.4	3.8	18.7	37	37	2.3	34.7

Table 21. Measured Strains for Static Test 10.

Gauge Number	Strains (Micro-Strain) at Wheel Positions							Strains (Micro-Strain)		
	1	2	3	4	5	6	7	Max	Min	Range
1	0.4	0.4	1.2	0				1.2	0	1.2
2	0.4	7.3	-1.1	-12.2				7.3	-12.2	19.5
3	0	-1.9	-1.9	-9.9				0	-9.9	9.9
4	0	-31	1.9	-9.6				1.9	-31	32.9
5	0	0.8	0.4	3.8				3.8	0.4	3.4
6	0	-0.4	1.9	5.4				5.4	-0.4	5.8
7	0	1.1	-0.7	-1.9				1.1	-1.9	3
8	0.4	2.7	3.5	-5.4				3.5	-5.4	8.9
9	-0.4	-1.5	-1.9	8.4				8.4	-1.9	10.3
10	0	-4.2	-5.7	-14.1				0	-14.1	14.1
11	0	-1.9	-1.1	-13				0	-13	13
12	0	1.9	4.2	-8.8				4.2	-8.8	13
13	0	-0.4	0	-7.3				0	-7.3	7.3

Dynamic Tests.

Eight series of dynamic tests were conducted for Gauge 4; this gauge is at the middle of the deck, see Fig. 3. A video camera was used to record dynamic test trucks' trucking companys and truck numbers; this information was used to obtain truck weights from the Fox scale. Dynamic strains at the middle of the deck, Gauge 4, are presented in Figs 10 and 11 and Table 22.

The purpose of this series of tests was to develop a methodology for recording dynamic strain data for future testing. Therefore, only limited test data were recorded in this series. Based on limited experimental data, the magnitude of the measured live load strains in the steel deck were very low. None of the dynamic test strain data provided strains as large as those recorded during static testing.

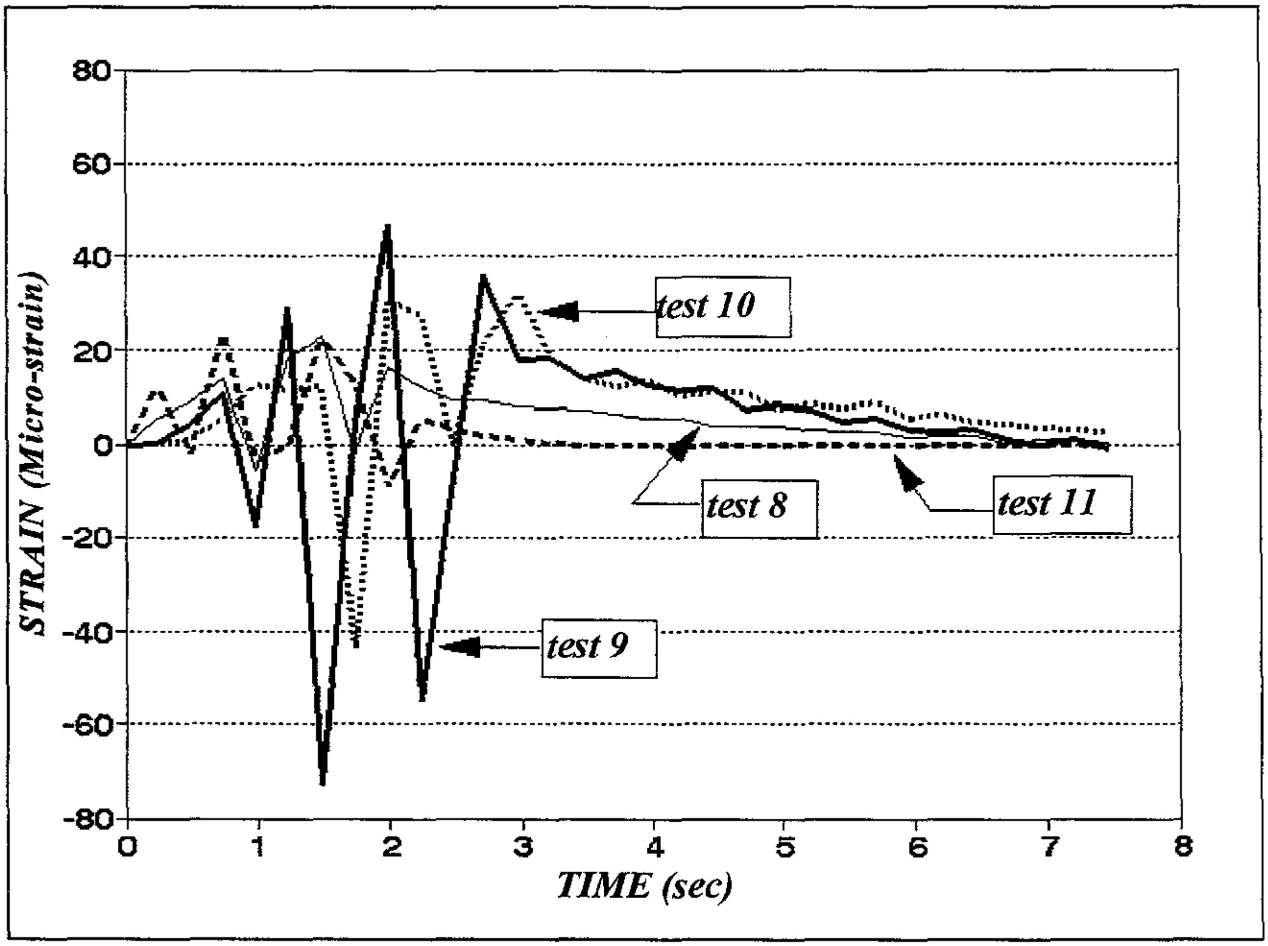

Fig. 10. Experimental Dynamic Strains, Gauge 4

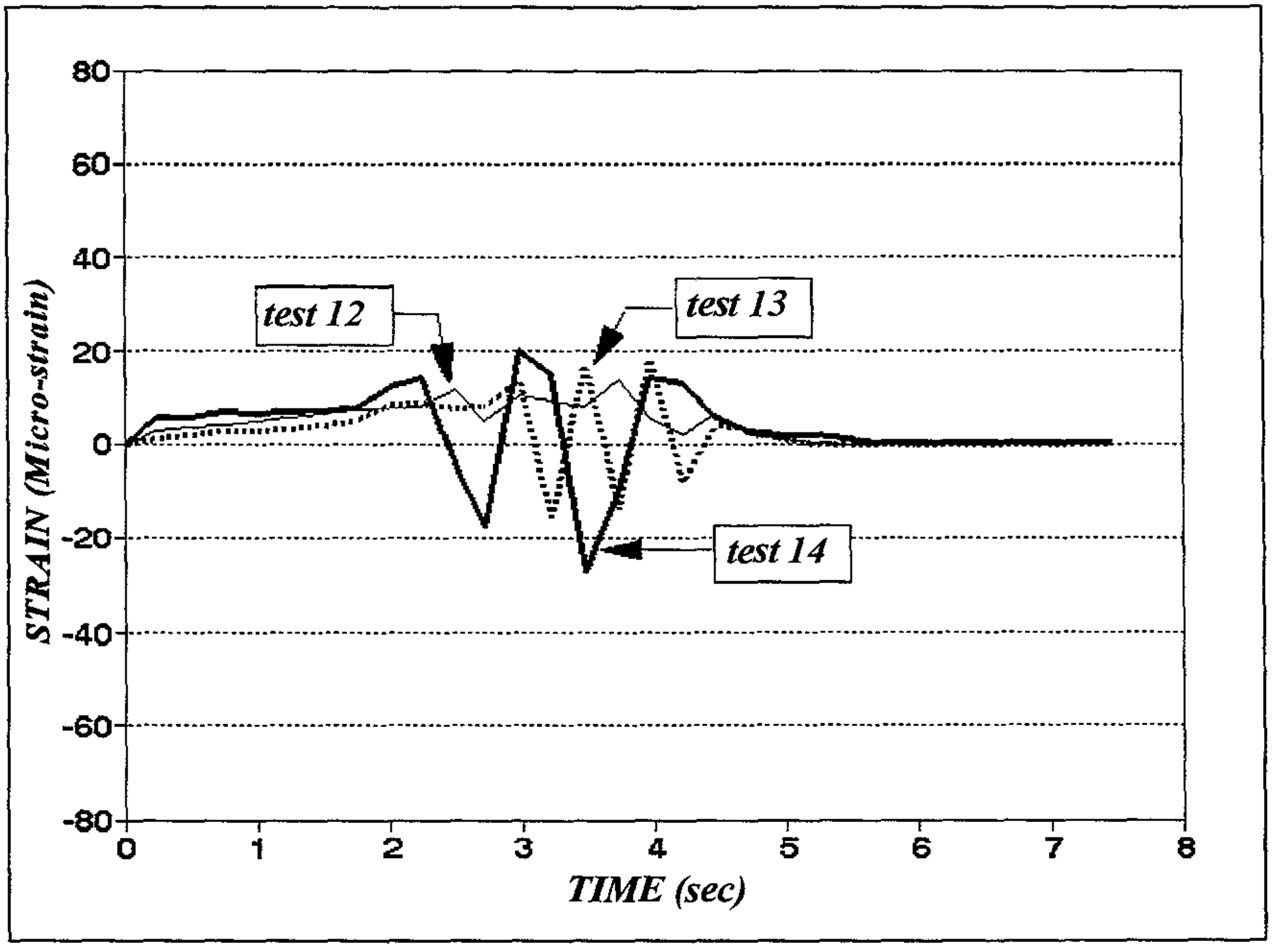

Fig. 11. Experimental Dynamic Strains, Gauge 4

Table 22. Dynamic Strain Measurements (Micro-Strain)

		Dynamic Tests							
Point #	Time (sec)	8	9	10	11	12	13	14	15
0	0	0	0	0	0	0	0	0	0
1	0.245	5.6	0.5	0.5	12.4	2.9	1.3	5.3	-1.8
2	0.497	9	4.5	1.3	-2.1	3.7	2.1	5.8	-1.8
3	0.745	13.8	10.5	5.6	24.1	4.2	2.8	6.8	-1.3
4	0.993	-5.8	-17.7	12.7	-3.4	5	2.9	6.6	-1.3
5	1.242	18.5	28.9	11.4	-0.3	5.8	3.4	6.9	-1
6	1.49	23	-72.8	12.2	23	6.1	4.2	7.4	-1
7	1.738	-1.1	4.8	-43.4	13.2	7.2	4.8	7.9	-0.8
8	1.987	16.4	46.4	31	-8.9	7.9	8.3	12.2	-0.8
9	2.235	12.2	-54.8	27.3	5.3	8.2	8.9	14.3	0
10	2.483	9.5	-7.2	0.3	3.5	11.9	7.7	-4.2	0
11	2.732	9.3	35.8	21.7	2.1	5	8.2	-17.7	0
12	2.98	8.2	18	32.5	0.8	10.6	14	19.9	1.1
13	3.228	7.7	18.5	18.4	0.3	9.5	-15.9	15.1	1.1
14	3.477	7.2	13.8	14	-0.3	8.2	17.2	-27.5	1.3
15	3.725	6.4	15.4	12.4	-0.5	14	-13.5	-9.3	1.9
16	3.973	5.6	12.7	13.8	-0.3	5.8	18.5	14.3	2.1

Table 22. Dynamic Strain Measurements (Micro-Strain) Continued

		Dynamic Tests							
Point #	Time (sec)	8	9	10	11	12	13	14	15
17	4.222	5.8	11.4	10.1	-0.5	1.9	-8	13.2	2.9
18	4.47	4.2	11.7	10.9	-0.5	6.1	4	5.3	2.9
19	4.718	4	7.4	10.9	-0.3	2.1	3.4	2.4	3.7
20	4.967	4.2	8.5	7.7	-0.5	1.1	0.8	2.1	4
21	5.215	3.2	7.4	8.5	-0.3	0.3	-0.3	1.9	4.2
22	5.463	2.6	5	7.9	-0.5	0	-0.3	0.8	5
23	5.712	2.5	5.6	8.9	-0.5	-0.3	-0.3	0.5	5.3
24	5.96	1.5	3.4	5.8	-0.5	0	-0.3	0.3	5.8
25	6.208	1.6	2.9	6.1	-0.3	0	0	0.5	8.1
26	6.457	1.9	3.2	5	-0.3	0	0	0.3	6.8
27	6.705	0.5	0.8	4.5	-0.3	0	0	0.3	7.7
28	6.953	1.1	-0.3	3.7	-0.5	0	-0.3	0.3	8
29	7.202	0.8	1.1	3.4	-0.5	0	-0.3	0.3	8.2
30	7.45	0.3	-1.1	2.9	-0.5	0	-0.3	0.5	9
Max	----	23	46.4	32.5	24.1	14	18.5	19.9	9
Min	----	-5.8	-72.8	-43.4	-8.9	-0.3	-15.9	-27.5	-1.8
Range	----	28.8	119.2	75.9	33	14.3	34.4	47.4	10.8

ANALYSIS

The objective of this part was to compare analytical with experimental results and to predict the maximum, minimum, and range of strains for both the steel deck and the several possible wearing surfaces for actual truck loads. Three methods were chosen for the analysis: an approximate method, a finite strip method, and a finite element method.

Analytical Methods for Orthotropic Decks.

The literature shows that the available analytical methodology for calculating stresses in orthotropic steel deck bridges fall in four categories: approximate methods, exact methods, finite strip method, and finite elements using plate elements combined with beam elements (Bouwkamp and Powell, 1969; Heins, 1976; Manko, 1987; Chikata, Kido, and Hattori, 19??; Lakshmy, Kumra, and Shamar, 1989; Dulevski, 1989; Cheung, 1969; Wolchuk and Ostabpenko, 1992; Nigaub, 1987; Xanthakos, 1993; Heins and Firmage, 1979; AISC, 1963; Troitsky, 1989; Van Der Walt, 1989). Several computer programs from the University of California Berkeley were tested for these types of problems and are currently in use. The Berkeley program FINPLA2 was created by Meyer and Scordelis (1971), and ABAQUS by Hibbitt, Karen & Sorensen, Inc. Both programs were used to compare experimental strains with calculated results. Truck weights for the vehicles tested during test days were sought from the AKDOT&PF weight station scale in Fox; however, the Fox scale does not show a record of weights for some of the vehicles tested, see Tables 8 and 9.

Finite Element Method

ABAQUS (Hibbitt, Karlsson & Sorensen, Inc.) was chosen to perform a static elastic anlaysis of the Yukon River Bridge Deck. ABAQUS is available on the Arctic Regional SuperComputer, Crey. More than 8,000 nodes and 10,000 shell elements were used to model a 320 ft bridge span. The span analyzed in this study stretched between the south abutment and the first pier. A 60 ft section of this span was instrumentated, see Fig. 2. The box girders, diaphragms, floor beams and bridge deck were modeled, see Fig. 6. A shell element (S4R5) was chosen for the analysis. Some of the features of this doubly curved element include four nodes, reduced integration, hour glass control, and five degrees of freedom per node.

Two trucks (test trucks 2 and 5 in Table 8) and 11 different live load cases were studied. Two temperature extreme load conditions were examined also. These tests corresponded to summer and winter temperature extremes (Hulsey, 1993). The following list provides a summary of the studies conducted.

- Bridge deck without timber wearing surface:
  - a). fixed boundary condition at the pier
  - b). simple boundary condition at the pier
  - c). the bearing support at the abutment incorporated in the model;
- Bridge deck with timber wearing surface;
- Bridge self weight;
- Bridge deck with a composite 2", 3" and 4" wearing surfaces with different moduli;
- Selfweight and live load;
- Temperature extremes for the bridge deck with a 4" wearing surface and a modulus of E = 760 ksi.

Several observations were made during these studies. First, the results for strains in the steel deck, assuming fixed versus pinned boundary conditions at the pier, were insignificant. Second, the stiffness of the timber deck showed little influence on the results, implying that the timber deck acted as a wearing surface. Third, live load tensile strains and the live load range of strain in the wearing surfaces studied were low. Thermal stresses were high and should be studied further.

Finite Strip Analysis

The program FINPLA2 was used on the VAX 8700 computer at the University of Alaska Fairbanks to analyze the first 120 deck feet of the first span. A larger model was not possible due to the program's memory limitations. The bridge deck and box girders were modeled with plate elements. The floor beams were modeled with diaphragm elements. The model consisted of 2,300 plate elements and 1,800 diaphragm elements. The influence of the boundary conditions were investigated initially. The studies performed using this program consisted of the following:

1. Bridge steel deck
    a). fixed boundary condition at the north end of the model
    b). simple boundary condition at the north end of the model

2. Bridge steel deck with a timber wearing surface
    a). steel and timber composite deck system
        - fixed boundary condition
        - simple boundary condition

    a). steel and timber noncomposite deck system
        - fixed boundary condition
        - simple boundary condition

3. Self weight

The analytical results showed little difference between steel deck strains if the timber wearing surface acted as a composite versus a noncomposite deck system. This similarity implies that the timber stiffness had little affect on the magnitude of live load strains in the steel deck.

The program CURDI5 (Van Der Walt, 1989), available from University of California Berkeley, was tested as a part of the study after the program was modified for the VAX8700 computer at the University of Alaska Fairbanks. Becaused of computer limitations, only three floor beams, a longitudinal bridge deck length of 45 ft, were included. No further studies were performed with this program.

Analytical Maximum Deck Strains.

The deck was analyzed for maximum anticipated flexural live load strains using an approximate method (Heins, 1976; Heins and Firmage,1979; AISC,1963; and Troitsky, 1987). The method is based on substructuring to evaluate deck strains and stresses; the results are presented in Table 23.

Table 23. Maximum Deck Strains

		Stresses (strains) in Bridge Deck			
		Top of Deck		Bottom of Deck	
Location	Loading	σ (ksi)	ϵ (micro-strain)	σ (ksi)	ϵ (micro-strain)
HS20-44 Truck					
mid span	System	-3.88	-134	5.79	200
support	LL @ rigid FB	0.97	33	-1.44	-50
Test Trucks		-5.1	-176	7.6	262

Experimental Comparison.

Strains were calculated using two programs: FINPLA2 and ABAQUS. FINPLA2 was developed for bridges and is available from the University of California Berkeley. This program uses the finite strip method to approximate the structural system. ABAQUS is available on the UAF Super Computer. Analysis were performed for the following conditions:

- Steel deck without any wearing surface and subjected to truck loads tested statically in the field;
- Steel deck with timber wearing surface and subjected to truck loads tested statically in the field;
- Steel deck with a two-inch alternative wearing surface having a modulus of 760 ksi and subjected to the loads tested statically in the field;
- Maximum anticipated live load wearing surface strains for a two-inch alternative wearing surface that was assumed bonded to the steel deck;
- Alternative wearing surface and steel deck geometry for conducting laboratory tests;
- Anticipated range of live load wearing surface strains for a two-inch alternative wearing surface that was considered bonded to the steel deck; and
- Steel deck under self weight.

Static Test 2. An eight-axle 144,440 lb truck was traveling north from Fairbanks. The truck was stopped and positioned at seven different pre-marked positions on first section of the bridge deck, see Fig. 2. Gauge locations and their orientation are presented in Fig. 3. Both ABAQUS and FINPLA2 were used to compare experimental gage strains for the seven load positions. Comparative strains are shown in Table 24. For this test series, ABAQUS provided a closer comparison with experimental results than did FINPLA2.

Table 24. Comparison of Analytical and Experimental Steel Deck Strains for Static Test 2: Timber Deck

Gauge Position	Strains (micro-strain)								
	Experimental			ABAQUS			FINAPL2		
	Max	Min	Range	Max	Min	Range	Max	Min	Range
S1	5	1	4	-2	-25	23	4	-5	9
S2	47	12	35	-11	-49	38	-1	-8	7
S3	35	11	24	2	-33	35	-9	-33	24
S4	38	3	35	7	-25	32	-31	-134	103
S5	8	-19	28	40	-62	102	-3	-14	11
S6	-9	-44	36	-12	-31	19	-45	-92	46
S7	37	-9	46	73	-47	120	-8	-20	13
S8	39	17	22	-14	-33	19	209	3	206
S9	-2	-19	17	54	-81	134	-8	-21	13
S10	21	4	16	-8	-15	7	70	27	43
S11	30	11	19	-7	-16	9	12	3	10
S12	19	9	10	-8	-23	15	32	-46	78
S13	29	13	16	-13	-29	15	18	3	15
Max/Min	47	-44	46	72	-81	134	209	-134	206

Static Test 5. A five-axle 88,180 lb truck traveling north was stopped and moved to seven at-rest-positions, see Fig. 2. ABAQUS and FINPLA2 results were compared with the experimental results. Strain results for this test series showed that FINPLA2 provided better correlation with experimental results. Comparative results are shown in Tables 25 and 26. These results are graphically presented in Figs. 12 through 18. Figs 20 through 23 present contours showing the strains and stresses in the steel deck.

All Static Tests. Figs 16 through 18 present a comparison between experimental and analytical steel deck strains.

Summary. Both FINPLA2 and ABAQUS were used to model a portion of the bridge superstructure. FINPLA2 provided better strain correlation with static Test 5 and ABAQUS showed better correlation for Test 2. Because of the limitations of FINPLA2, additional strain predictions for alternative wearing surfaces were performed with ABAQUS. The results of these studies are presented in the following chapter.

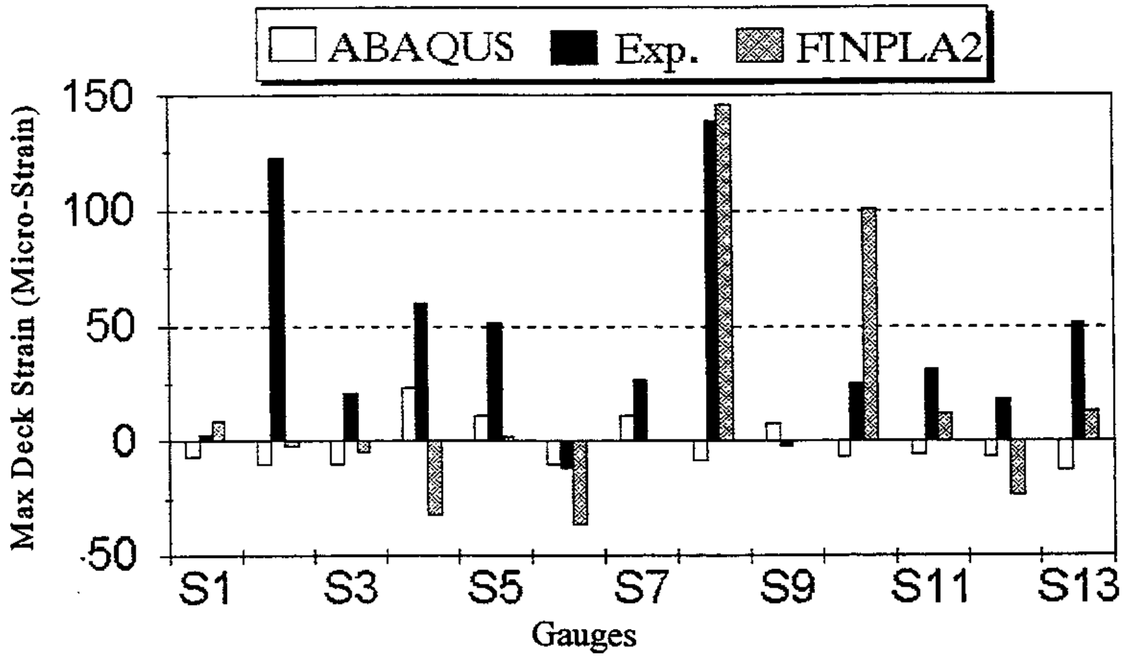

Fig. 12. Maximum Gauge Strains for Static Test 5

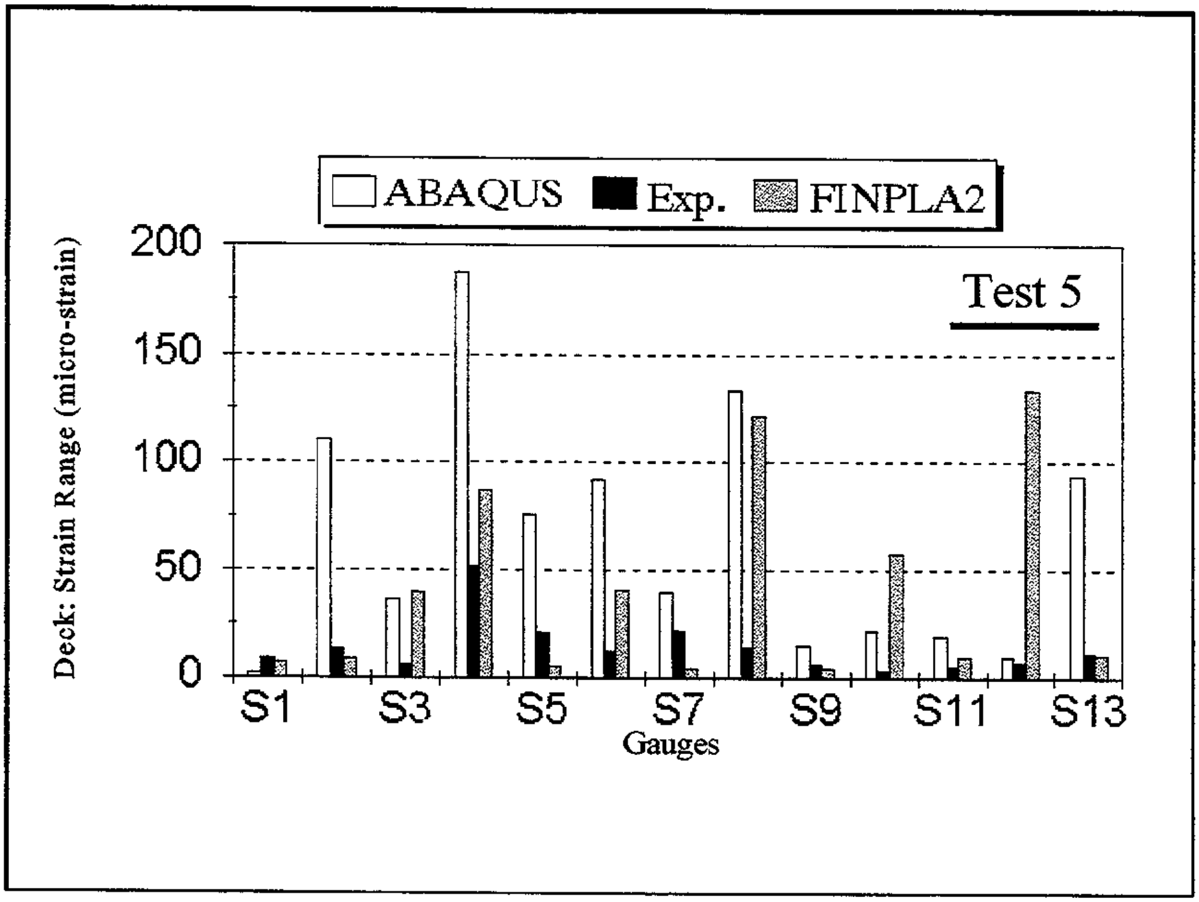

Fig. 13. Range of Steel Deck Strains for Test 5

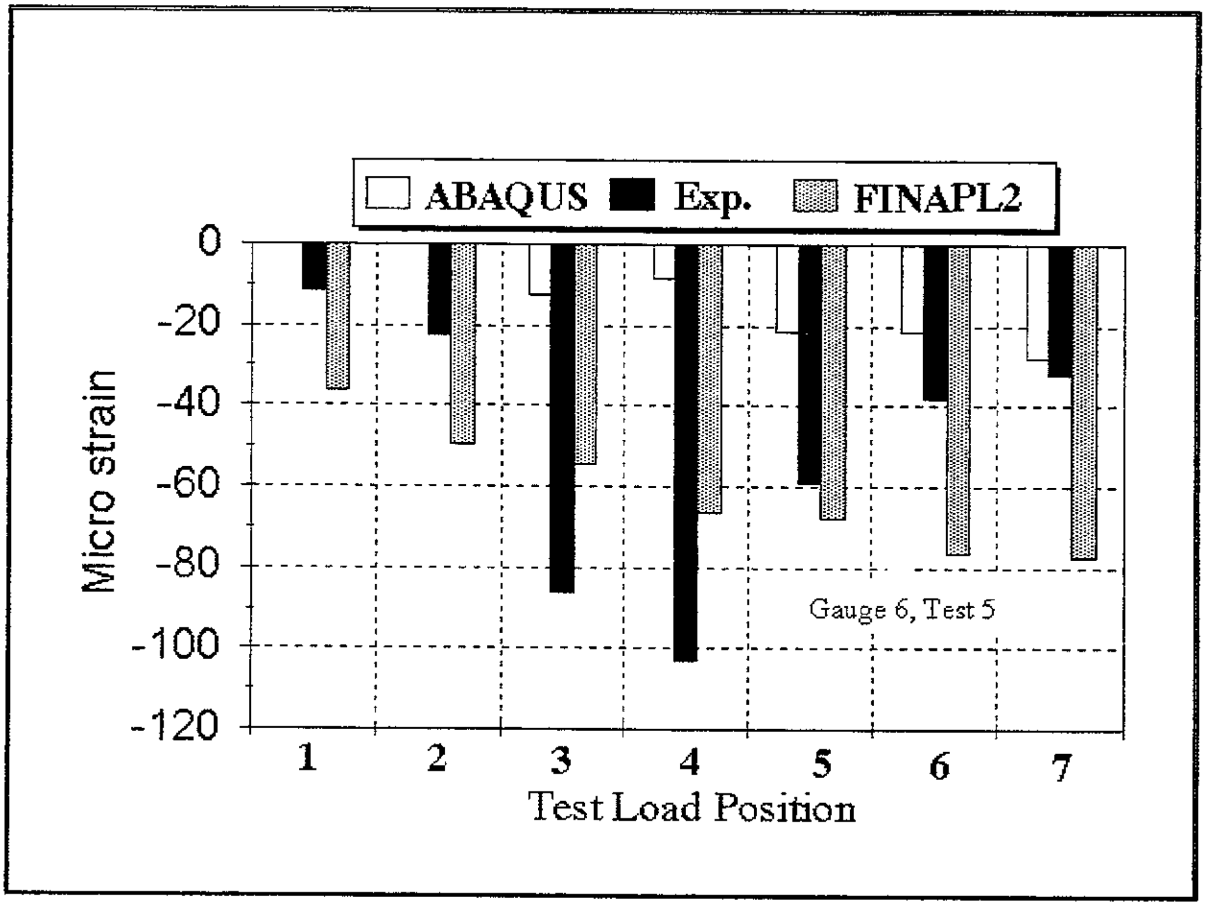

Fig. 14. Maximum Steel Deck Strains for Gauge 6

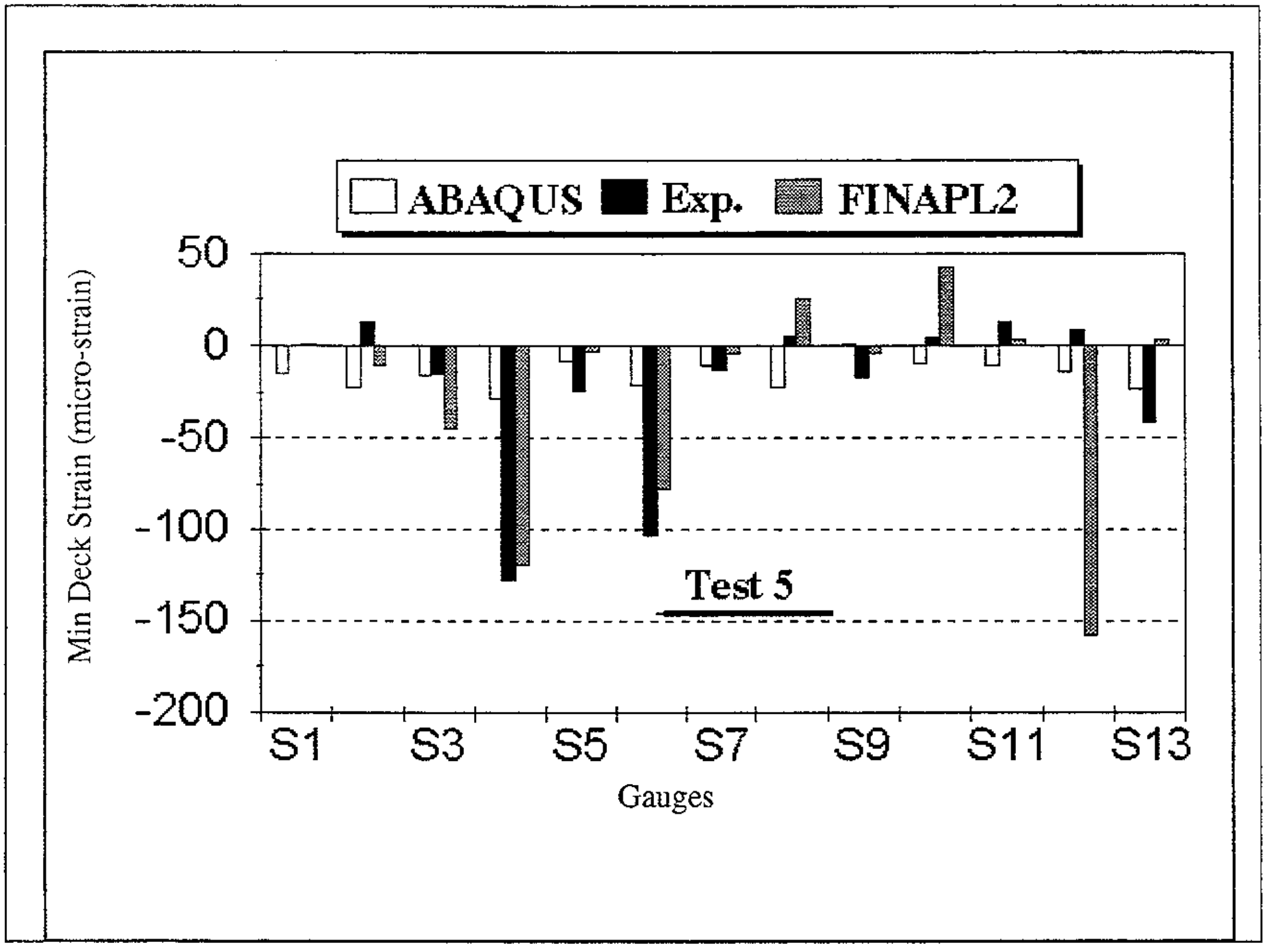

Fig. 15. Minimum Gauge Strains for Static Test 5

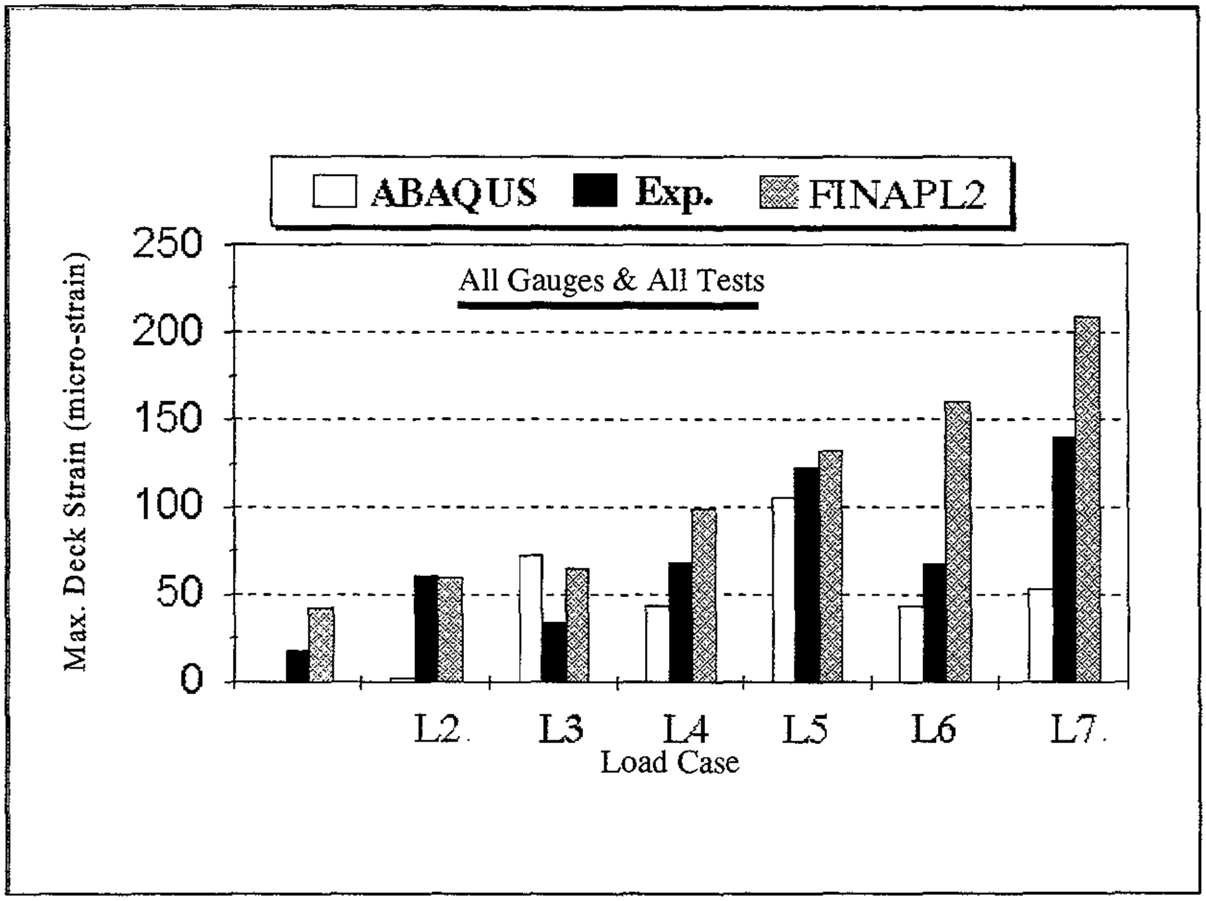

Fig. 16. Maximum Steel Deck Strains, All Static Tests

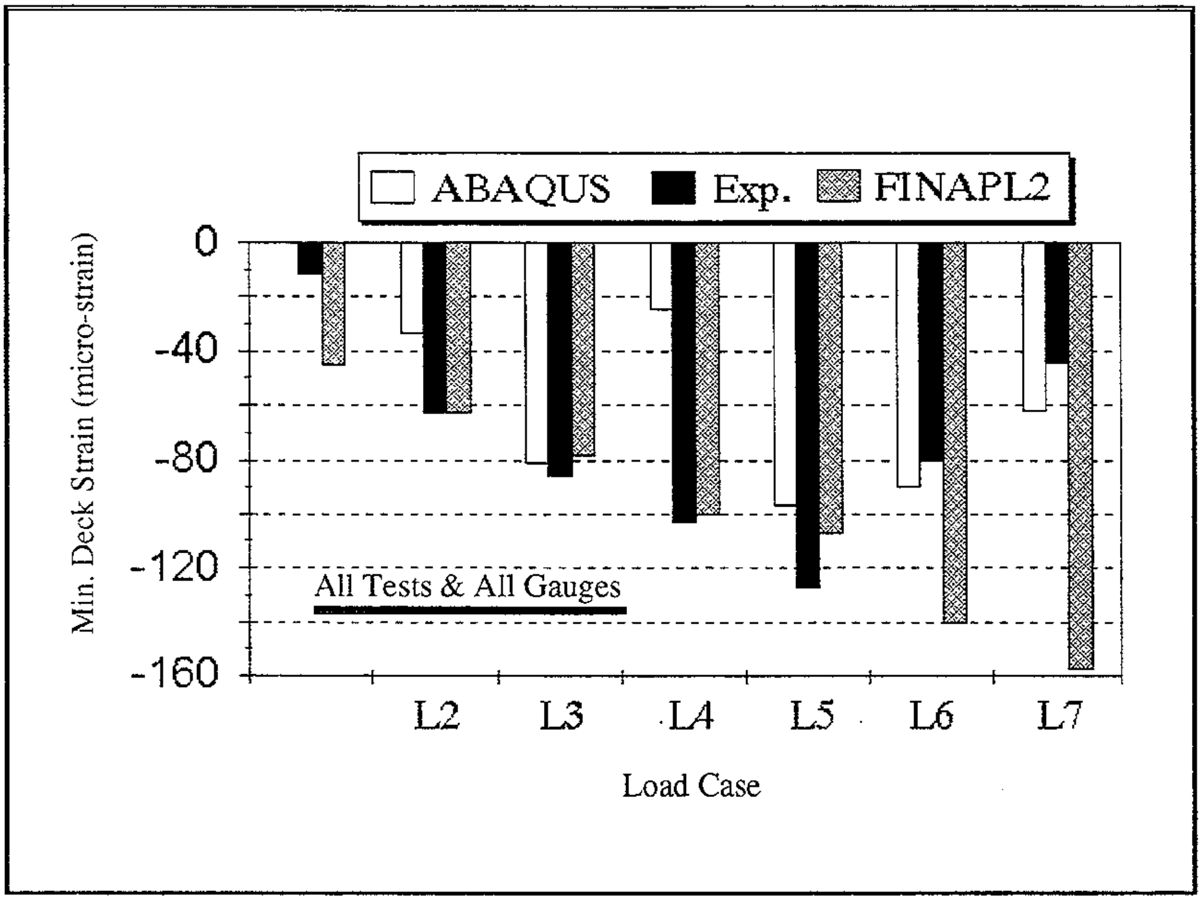

Fig. 17. Minimum Steel Deck Strains, All Static Tests

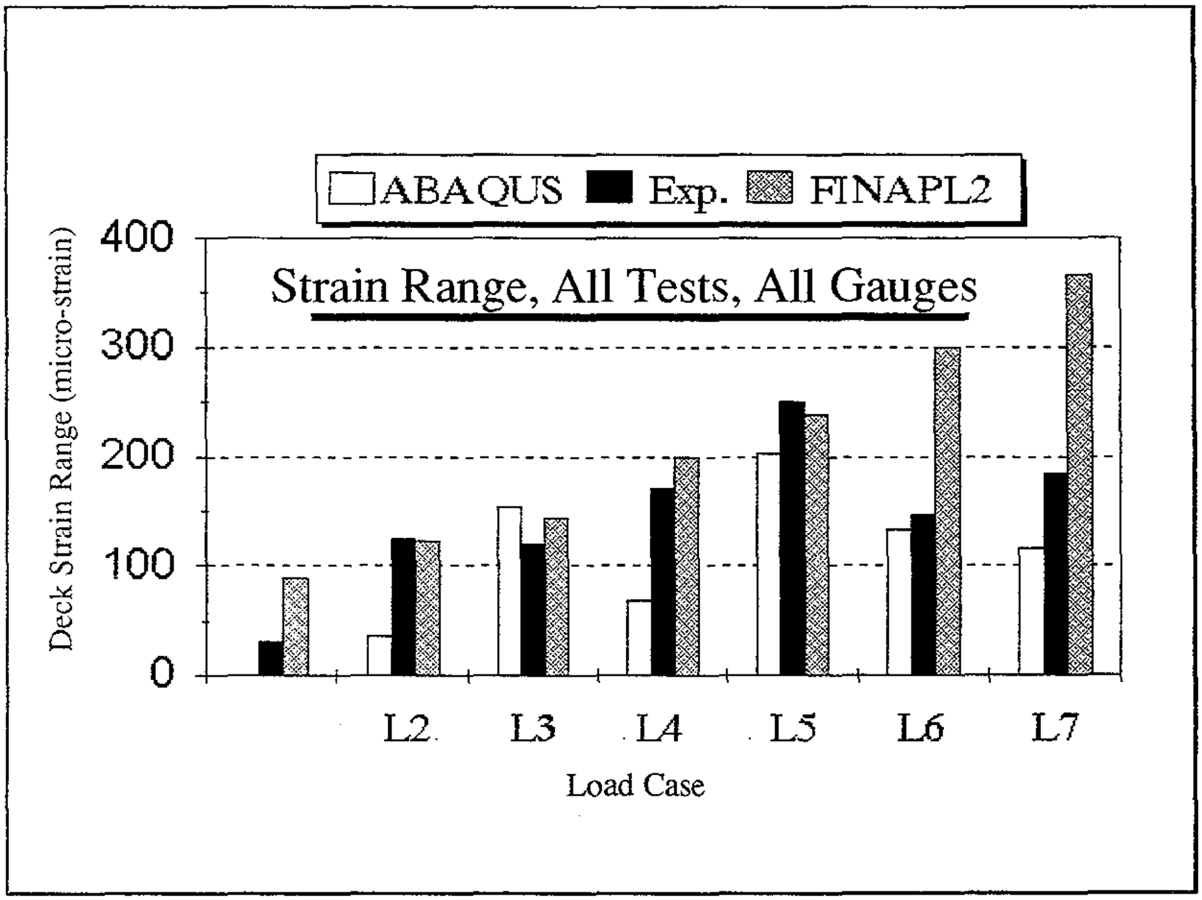

Fig. 18. Range of Steel Deck Strains, All Static Tests

Table 25. Comparison of Analytical and Experimental Steel Deck Strains for Test 5; With Timber Deck

Gauge Position	Strains (micro-strain)								
	Experimental			ABAQUS			FINAPL2		
	Max	Min	Range	Max	Min	Range	Max	Min	Range
S1	2	0	2	-7	-15	8	8	1	7
S2	123	13	110	-10	-23	13	-2	-11	9
S3	21	-15	36	-10	-16	6	-5	-45	40
S4	60	-127	187	24	-29	52	-32	-119	87
S5	52	-24	76	11	-9	20	1	-3	5
S6	-11	-103	92	-10	-22	12	-36	-77	41
S7	27	-13	40	11	-10	21	0	-4	4
S8	139	5	134	-8	-22	13	146	25	121
S9	-2	-17	15	7	1	6	0	-4	4
S10	25	4	21	-7	-10	3	101	43	58
S11	31	12	19	-6	-10	5	12	3	9
S12	18	9	10	-7	-14	7	-24	-157	134
S13	52	-42	93	-13	-24	11	13	3	10
Max/Min	139	-127	187	24	-28	52	146	-157	134

Table 26. Analytical Strain Comparison at All Gauges for Each Load Case, Test 5

Load Case	By Experiment			By ABAQUS			By FINAPLA2		
	Max	Min	Range	Max	Min	Range	Max	Min	Range
L1	18	-12	30	----	-----	-----	43	-45	88
L2	61	-63	123	3	-33	36	59	-63	122
L3	34	-86	120	73	-81	153	65	-79	143
L4	68	-103	171	44	-25	68	99	-100	199
L5	123	-127	250	105	-97	202	132	-107	239
L6	67	-80	147	43	-90	133	159	-140	299
L7	139	-44	183	54	-62	116	209	-157	366

ANALYSIS FOR ALTERNATIVE WEARING SURFACES

Alternative wearing surfaces for the Yukon River Bridge deck were examined using ABAQUS. A portion of the bridge superstructure was modeled. The model incorporated the abutment pinned bearing support, box girders, floor beams, steel deck and various bonded wearing surfaces. The information presented herein was developed to assist design engineers in selecting possible alternative wearing surfaces for the Yukon River Bridge Deck. The information presented does not account for traction or abrasion.

Existing Conditions

The Yukon River Bridge has a temporary timber deck wearing surface. The timber deck has 2 layers of 3" x 12" boards; the top layer is transverse to traffic. The finished thickness of each layer is approximately 2 1/2" thick. For purposes of this study, the modulus of the timbers were assumed to be 1,600 *ksi*. Strains and stresses in the steel deck were calculated for the instrumentated section between floor beams 3 and 4. This section is shown in Figs. 3 and 4. Both maximum and minimum live load strains and stresses in the steel deck with the temporary timber wearing surface are shown in Table 27. The maximum live load stresses in the steel deck were 1,665 psi. Table 28 shows the maximum calculated stresses in the steel deck for static Tests 2 and 5 if the timber deck was removed. The maximum live load stresses in the steel deck for this condition were 9,761 psi. The minimum live loads stresses in the steel deck were -11,625 psi. The range of stress was 21,386 psi for the instrumentated section. The maximum stresses in the steel plate with the timber deck was 3,814 psi, see Table 29. The calculated live load stresses in

Table 27. Calculated Maximum and Minimum Steel Deck Strains and Stresses for Timber Deck Wearing Surface[a]

Test	Load	Transverse Direction						Longitudinal Direction					
		Strains (micro-strain)			Stresses (psi)			Strains (micro-strain)			Stresses (psi)		
		Max	Min	Range	Max	Min	Range	Max	Min	Range	Max	Min	Range
2	2	31	-114	145	748	-401	1,149	56	-208	264	89	-1,266	1,355
	3	58	-116	174	1,665	-716	2,381	59	-195	254	253	-1,171	1,424
	4	55	-37	92	142	-663	805	42	-56	98	68	-983	1,051
	5	28	-102	130	783	-446	1,229	66	-210	276	240	-978	1,218
	6	50	-46	96	567	-595	1,162	48	-91	139	77	-970	1,047
	7	64	-65	129	885	-701	1,586	47	-103	150	76	-817	893
5	3	51	-24	75	236	-501	737	28	-65	93	46	-756	802
	4	92	-29	121	214	-738	952	76	-111	187	122	-1,221	1,343
	5	111	-110	221	1,559	-881	2,440	86	-170	256	273	-1,316	1,589
	6	43	-103	146	1,249	-522	1,771	43	-153	196	69	-1,002	1,071
	7	14	-42	56	215	-270	485	18	-42	61	29	-997	1,026
2,5	All	111	-116	221	1,665	-881	2,440	86	-210	276	273	-1,316	1,589

[a]Values for deck section instrumentated between floor beams 3 and 4.

Table 28. Calculated Maximum and Minimum Steel Deck Strains and Stresses without Timber Deck Wearing Surface- Instrumentated Section[a]

Test[c]	Load Case[b]	Transverse Direction						Longitudinal Direction					
		Strains (micro-strain)			Stress (psi)			Strains (micro-strain)			Stresses (psi)		
		Max	Min	Range	Max	Min	Range	Max	Min	Range	Max	Min	Range
2	2	74	-104	178	2,182	-3,875	6,057	181	-178	360	6,084	-6,398	12,482
	3	231	-224	456	7,278	-7,893	15,171	280	-244	524	9,559	-8,647	18,206
	4	78	-86	164	2,519	-3,800	6,319	184	-150	334	6,047	5,067	980
	5	100	-133	233	2,688	-4,706	7,394	85	-98	183	2,921	-3,177	6,098
	6	130	-119	249	4,025	-4,203	8,228	185	-178	363	6,375	-6,341	12,716
	7	125	-142	267	6,055	-6,583	12,638	254	-199	453	9,437	-7,668	17,105
5	3	83	-70	153	3,738	-3,037	6,775	202	-133	334	6,531	-4,846	11,377
	4	138	-142	280	4,902	-4,525	9,427	162	-88	250	5,018	-3,458	8,476
	5	262	-248	510	8,331	-8,701	17,032	167	-124	291	5,478	-5,565	11,043
	6	306	-343	649	9,761	-11,625	21,386	207	-218	425	6,440	-6,959	13,399
	7	109	-124	233	3,670	-4,627	8,297	42	-86	127	2,080	-3,517	5,597
2,5	All	306	-343	649	9,761	-11,625	21,386	281	-243	523	9,559	-8,647	18,206

[a]Calculations for deck section instrumented between floor beams 3 and 4.

[b]Position of front wheels for test truck.

[c]Test truck 2 weighed 114,400 lbs, see Table 8

Table 29. Calculated Maximum and Minimum Steel Deck Strains and Stresses with Timber Deck Wearing Surface[a]

Test	Load Case	Transverse Direction						Longitudinal Direction					
		Strains (micro-strains)			Stresses (psi)			Strains (micro-strains)			Stresses (psi)		
		Max	Min	Range	Max	Min	Range	Max	Min	Range	Max	Min	Range
2	2	116	-180	296	2,507	-1,083	3,590	91	-254	345	484	-1,300	1,784
	3	58	-116	174	1,165	-1,215	2,380	102	-195	297	401	-1,171	1,572
	4	81	-199	280	2,628	-952	3,580	102	-297	399	617	-983	1,600
	5	162	-184	346	2,507	-1,307	3,814	131	-266	397	779	-1,237	2,016
	6	50	-81	131	746	-595	1,341	68	-187	255	282	-970	1,252
	7	122	-178	300	2,517	-1,044	3,561	94	-254	348	603	-1,060	1,663
5	2	74	-102	176	1,609	-1,060	2,669	91	-167	258	814	-847	1,661
	3	92	-138	230	2,020	-980	3,000	76	-187	263	405	-1,221	1,626
	4	116	-110	226	1,559	-881	2,440	86	-170	256	273	-1,316	1,589
	5	79	-142	221	1,902	-759	2,661	66	-187	253	321	-1,016	1,337
	6	55	-114	169	1,586	-636	2,222	55	-172	227	348	-1,128	1,476
2,5	All	162	-199	346	2,628	-1,307	3,814	131	-297	399	814	-1,316	2,016

[a]Strain values for all of the deck

the deck without the timber wearing surface would be 15,400 psi with a range of 31,010 psi, see Table 30. The transverse and longitudinal strains and stresses in the steel deck for static test truck 5 are presented in Figs. 20 through 23.

Self Weight

Strains and stresses were calculated for the self weight of the steel deck with the existing timber deck. The strains and stresses at each gauge position were calculated for the self weight, see Table 31. The maximum state of stress in the steel deck due to self weight was -1,948 psi. The displacement profile for the first span is shown in Fig. 19.

Strains for Two-Inch Wearing Surface

Assume the temporary two-layer timber deck wearing surface on the Yukon River Bridge is removed and replaced with another wearing surface. The engineer will want to know the magnitude of the live load strains and stresses in the proposed wearing surface mand whether or not cold weather temperature stresses would be detrimental to the proposed wearing surface. This part of the study attempted to provide answers to these questions for a given wearing surface.

Strains and stresses were examined for a two-inch future wearing surface and are shown in Figs. 24 through 27. The wearing surface material modulus chosen for study was 760 ksi. Under these conditions, live load strains in the wearing surface were small. Summer and winter temperature extremes similar to Fairbanks weather (Hulsey and Powell, 1993) were used for thermal loading, see Fig. 28. The resulting thermal strain and stress contours within the wearing surface are shown in Figs. 29 and 30. The calculated thermal stresses were extremely high.

Table 30. Calculated Maximum and Minimum Steel Deck Strains & Stresses without Timber Deck Wearing Surface[a]

Test	Load Case	Transverse Direction						Longitudinal Direction					
		Strains (micro-strain)			Stresses (psi)			Strains (micro-strain)			Stresses (psi)		
		Max	Min	Range	Max	Min	Range	Max	Min	Range	Max	Min	Range
2	2	382	-390	772	12,700	-13,600	26,300	252	-244	496	8,514	-11,260	19,774
	3	232	-224	456	7,278	-7,893	15,171	281	-244	525	9,559	-8,647	18,206
	4	473	-454	927	15,400	-15,610	31,010	183	-178	361	6,095	-8,959	15,054
	5	425	-454	879	14,240	-16,000	30,240	151	-281	432	7,462	-13,300	20,762
	6	130	-120	250	4,025	-4,203	8,228	185	-178	363	6,373	-6,341	12,714
	7	405	-415	820	13,520	-14,280	27,800	254	-242	496	9,437	-9,926	19,363
5	2	416	-451	867	13,860	-14,890	28,750	201	-132	333	6,531	-6,111	12,642
	3	266	-310	576	8,613	-10,400	19,013	161	-128	289	5,010	-5,423	10,077
	4	327	-373	700	10,700	-12,560	23,260	167	-161	328	5,478	-5,599	10,433
	5	306	-343	649	9,761	-11,625	11,625	208	-218	426	6,440	-6,959	13,399
	6	324	-401	725	10,800	-13,500	24,300	65	-119	184	3,778	-5,437	9,215
2,5	All	473	-454	927	15,400	-16,000	31,010	281	-281	525	9,559	-13,300	20,762

[a] Strain values for all of the deck

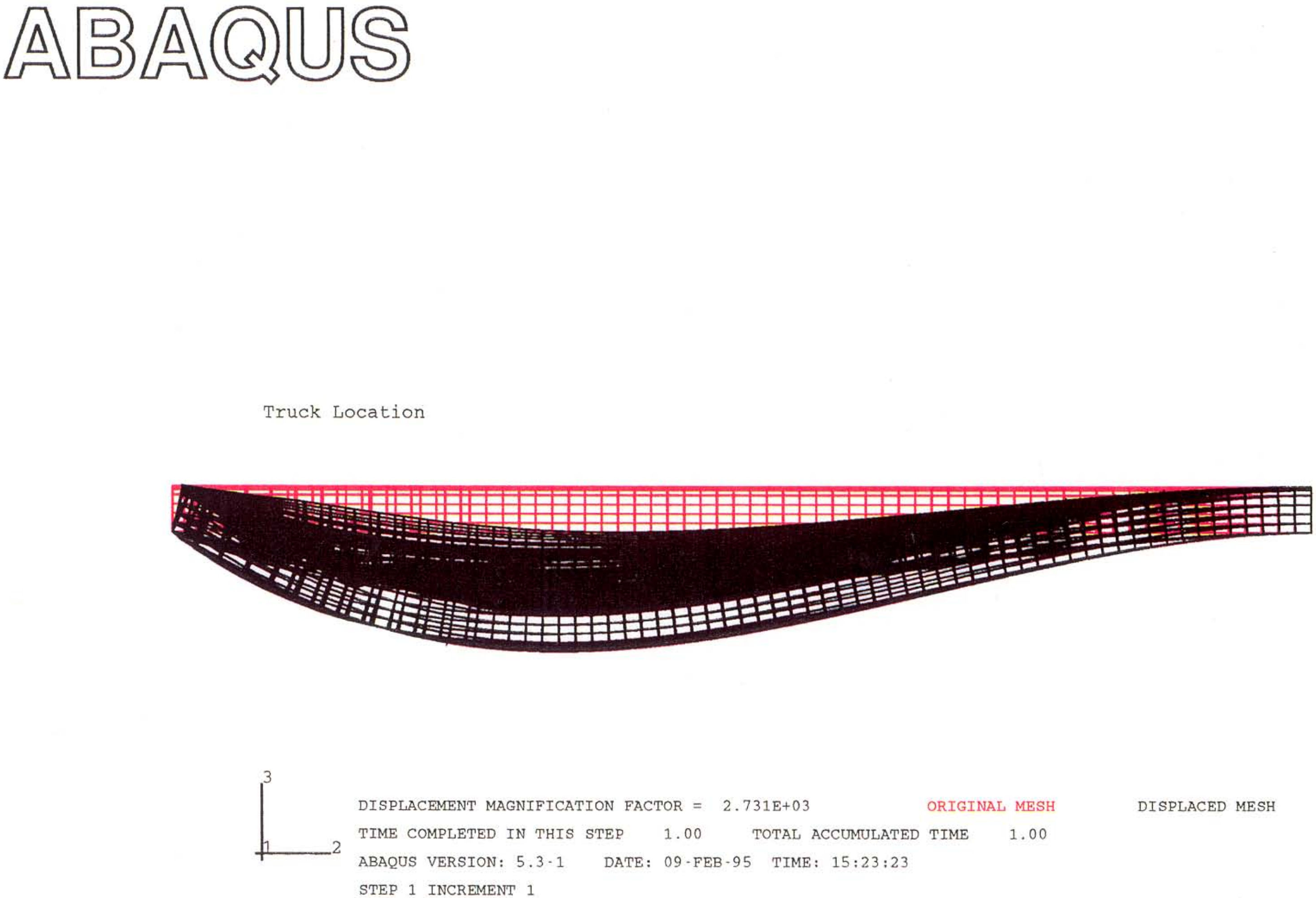

Fig. 19 Self Weight Displacement Profile of Steel Deck, 1st Span

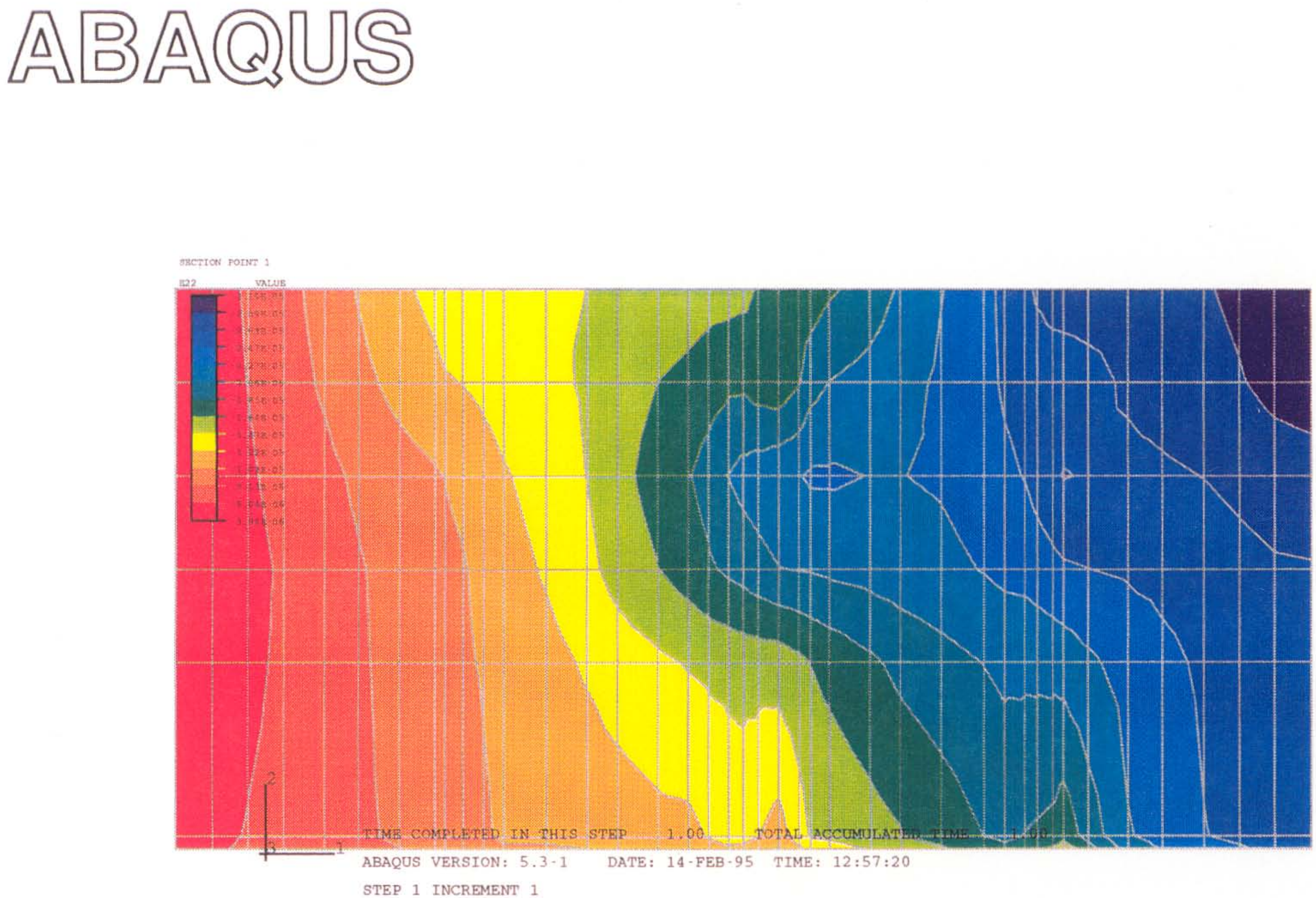

Fig. 20 Contour of Transverse Steel Deck Strains for Test 5 and a Timber Deck

ABAQUS

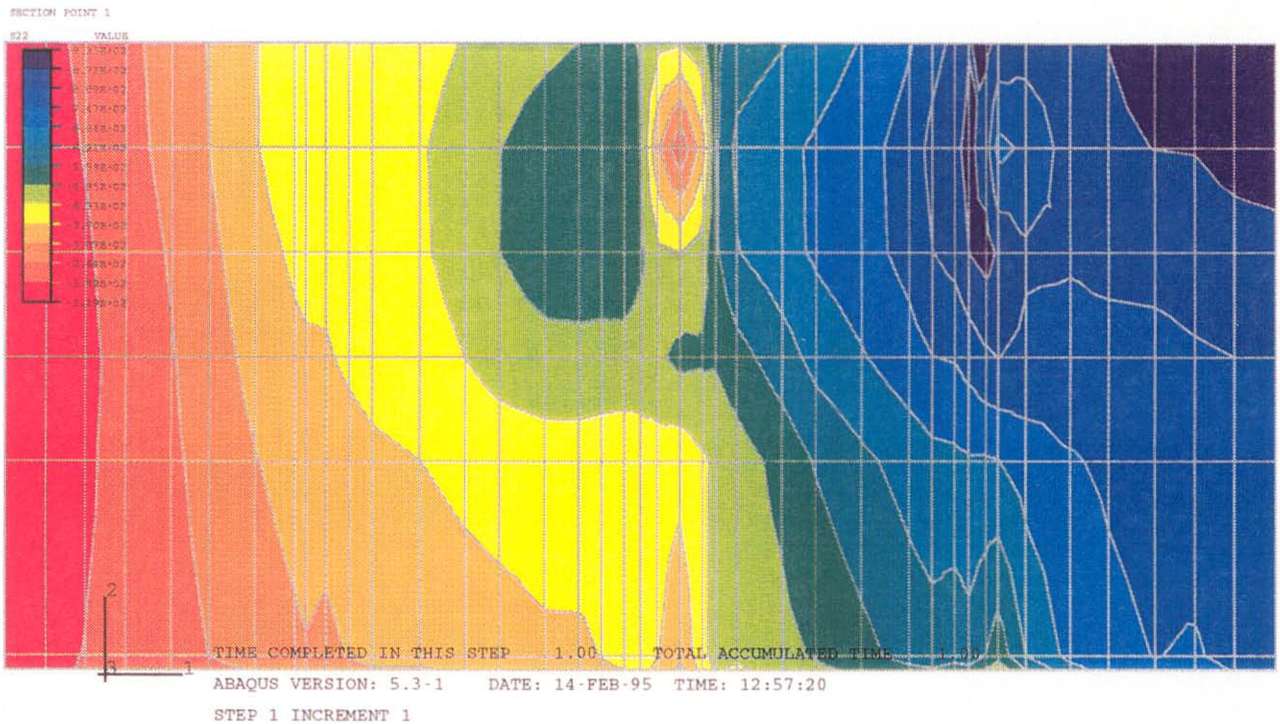

Fig 21. Contour of Transverse Steel Deck Stresses for Test 5 and a Timber Deck

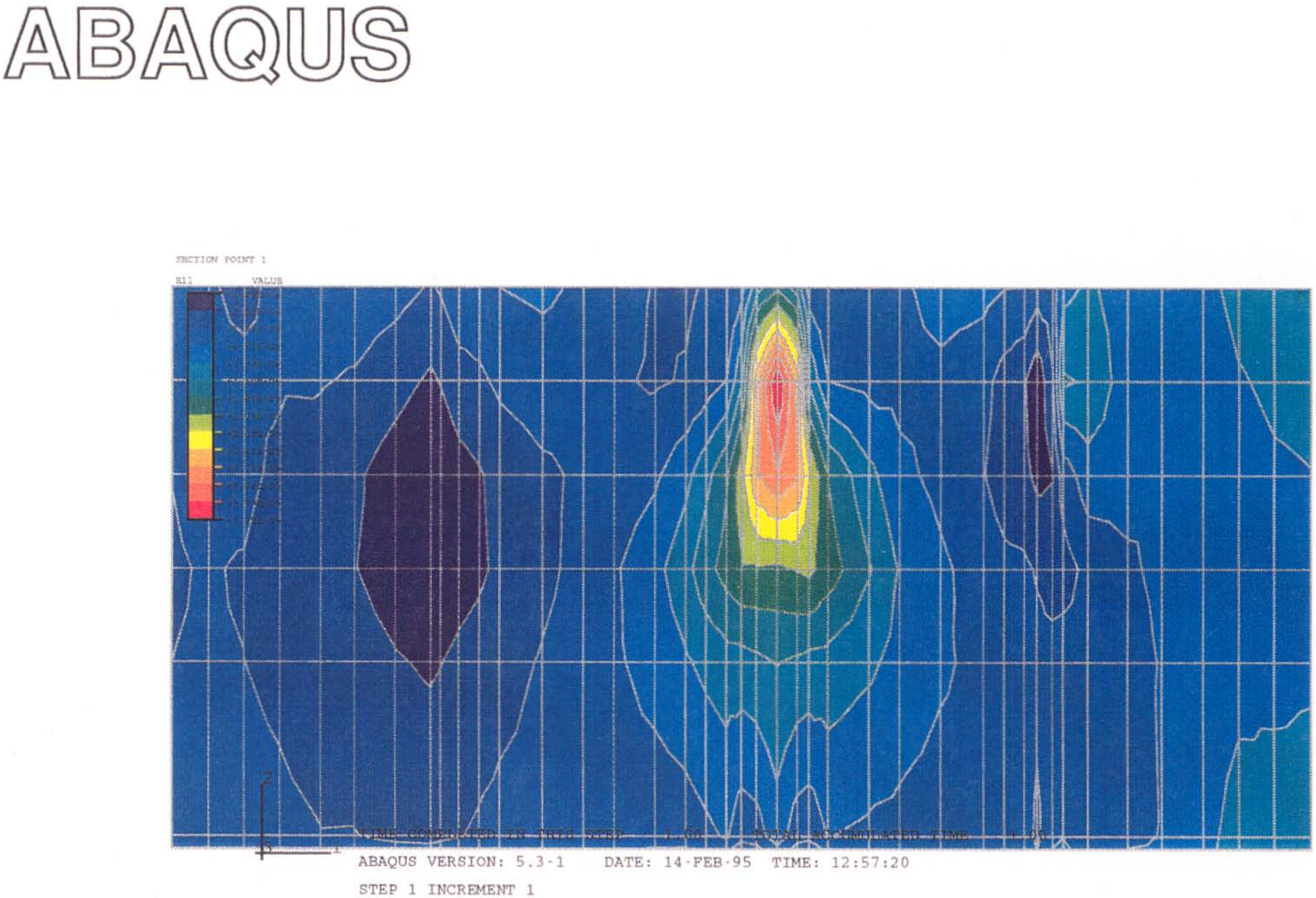

Fig 22 Contour of Longitudinal Steel Deck Strains for Test 5 and a Timber Deck

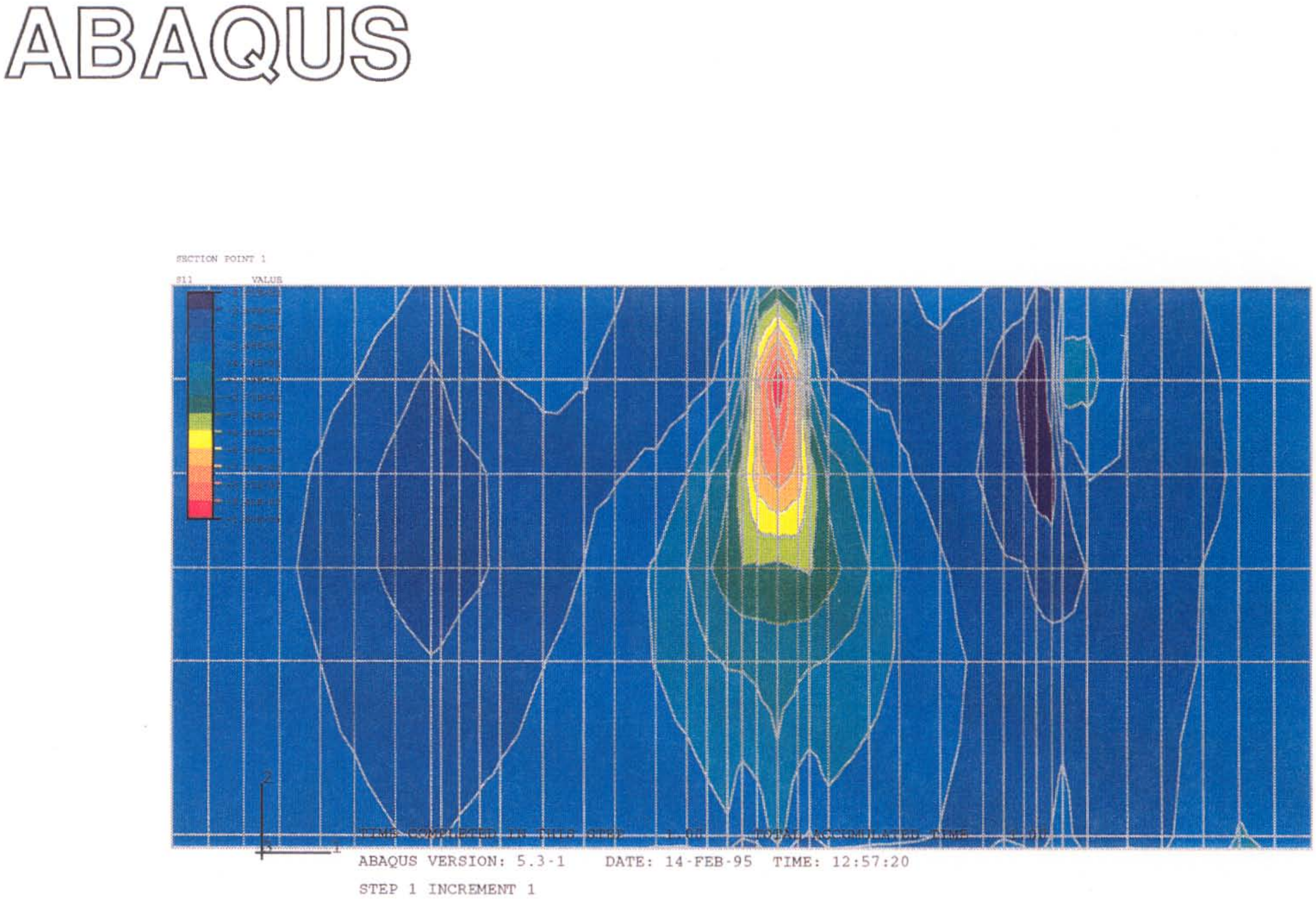

Fig. 23 Contour of Longitudinal Steel Deck Stresses for Test 5 and a Timber Deck

Table 31. Calculated Steel Deck Strains and Stresses for Self Weight

Gauge Position	Strain (micro-strain)	Stress (psi)
S1	-37	-1,178
S2	-47	-1,463
S3	-36	-1,198
S4	-37	-1,145
S5	-1	-55
S6	-4	-1,232
S7	2	-397
S8	-60	-1,948
S9	5	-225
S10	-40	-1,159
S11	-46	-1,364
S12	-45	-1,325
S13	-39	-1,197

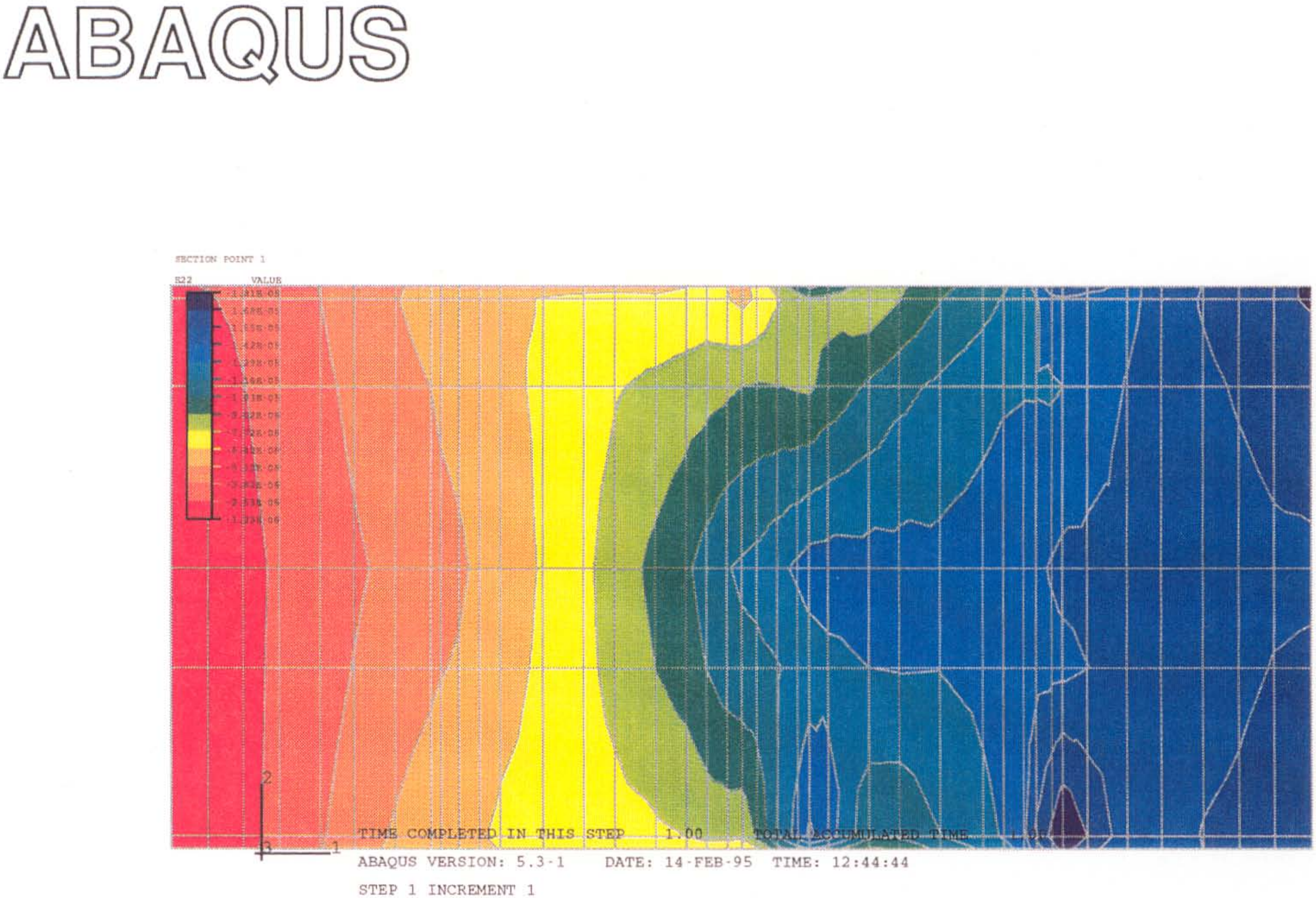

Fig. 24 Contour of Transverse Strains in a 2-in Wearing Surface (E_p = 760 ksi), Test 5

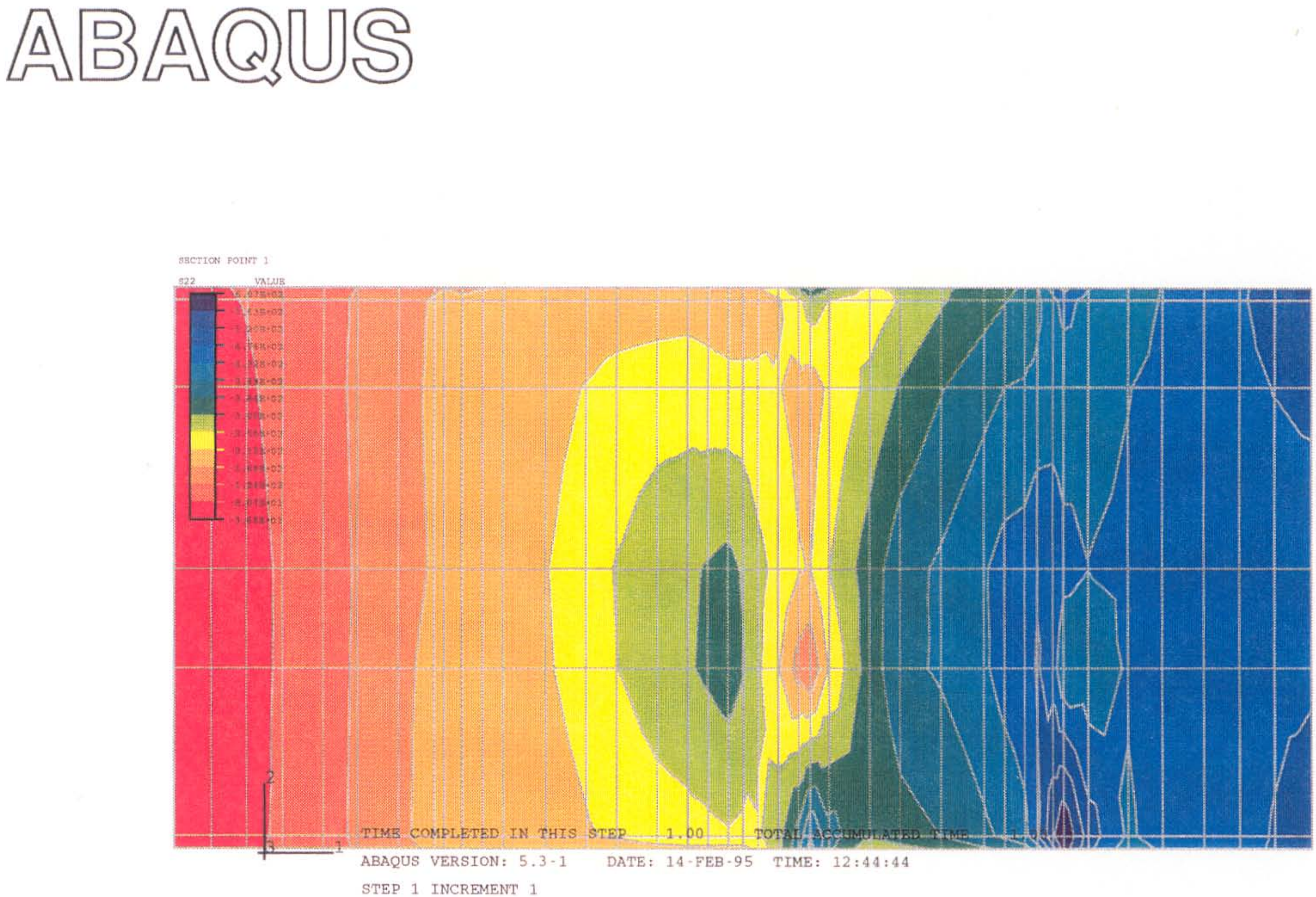

Fig. 25 Contour of Transverse Stresses in a 2-in Wearing Surface (E_p = 760 ksi), Test 5

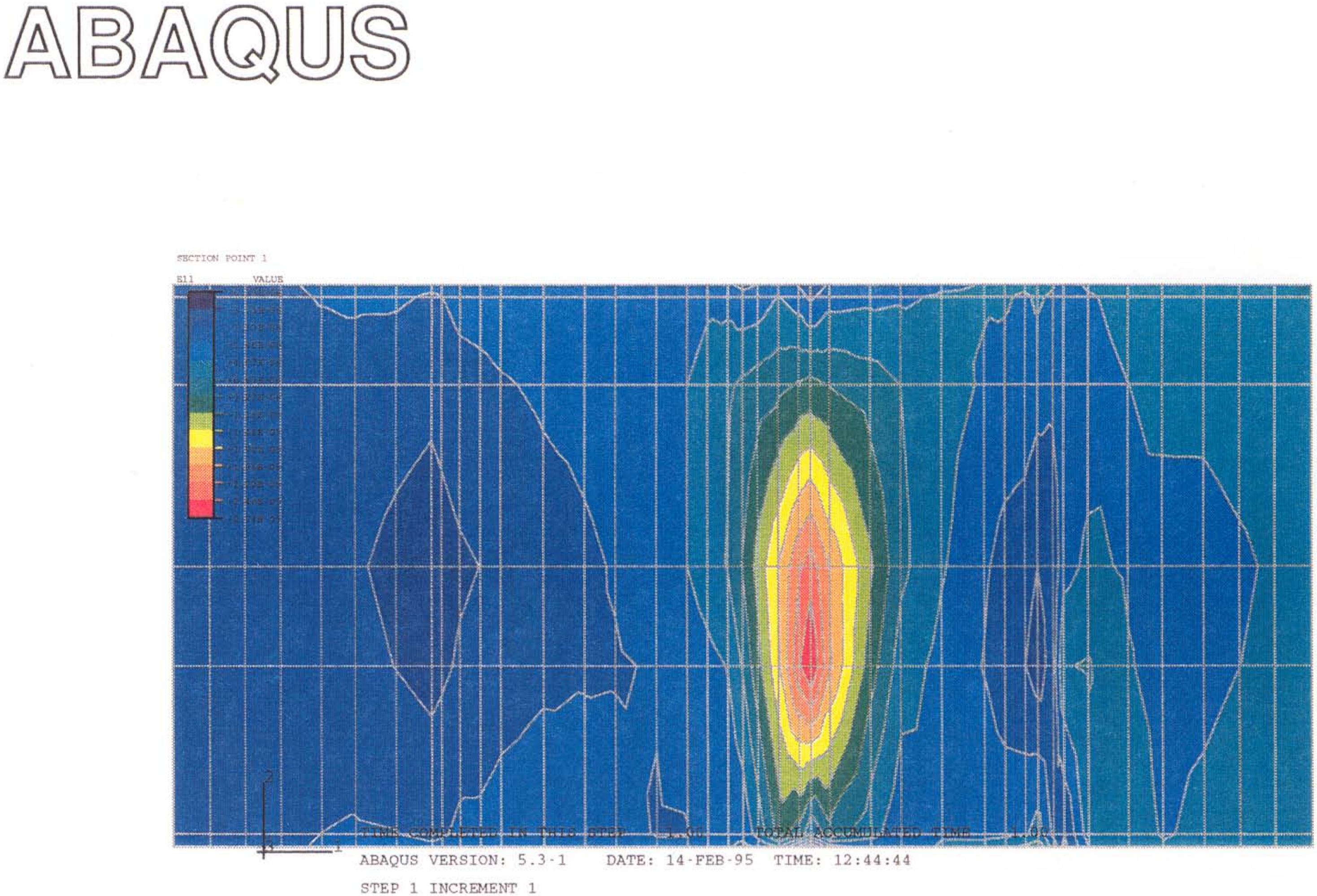

Fig. 26 Contour of Longitudinal Strains in a 2-in Wearing Surface (E_p = 760 ksi), Test 5

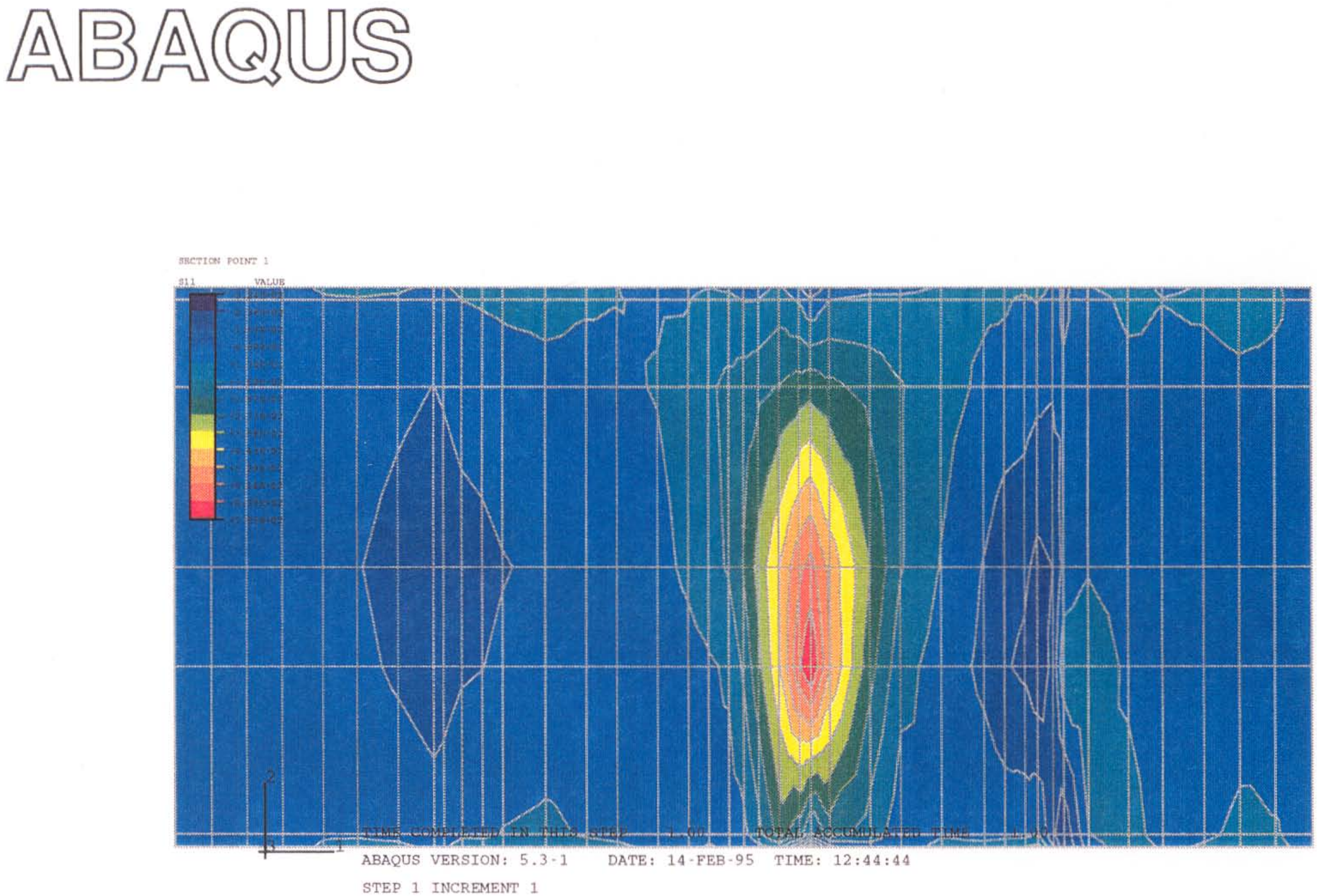

Fig. 27 Contour of Longitudinal Stresses in a 2-in Wearing Surface (E_p = 760 ksi), Test 5

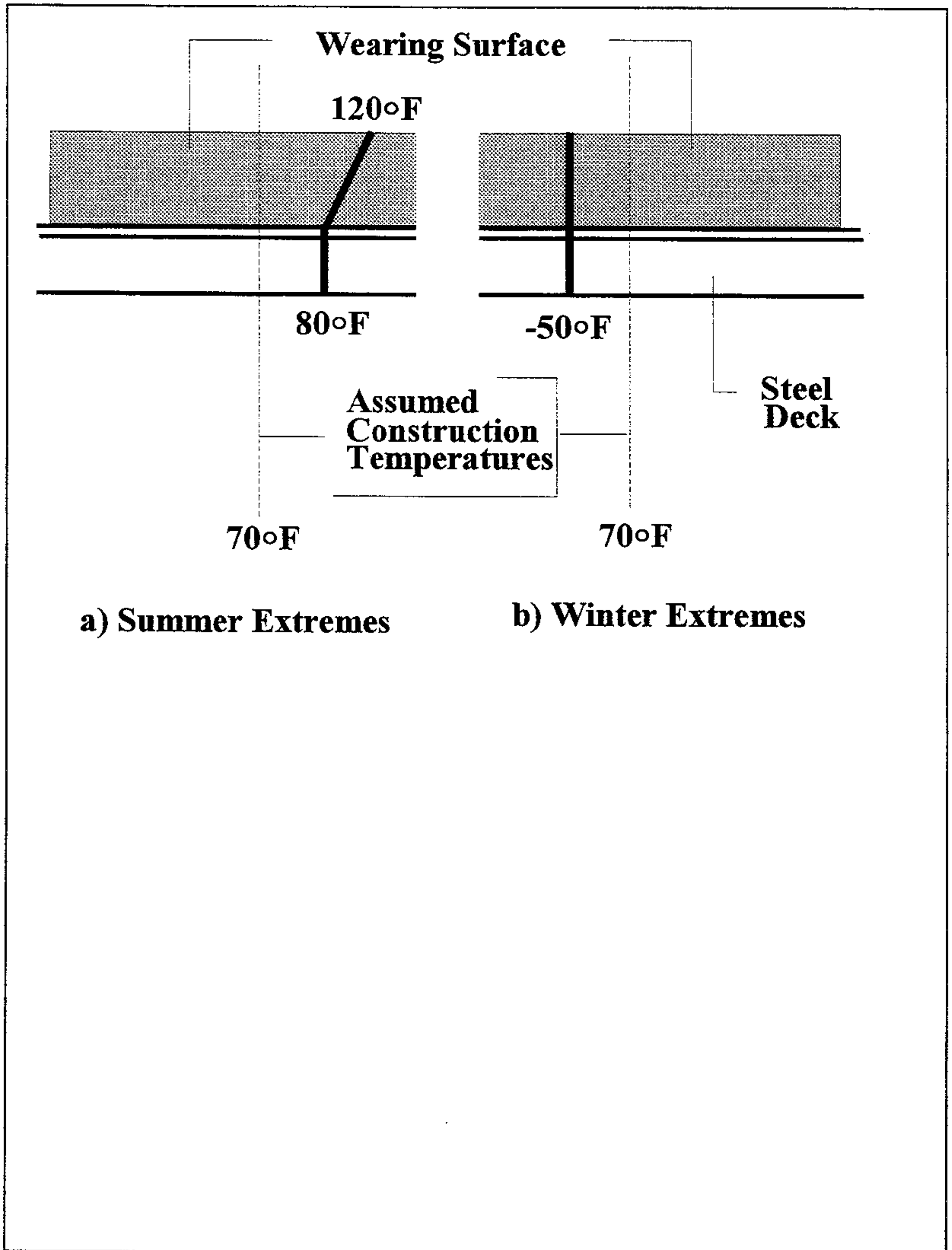

Fig. 28. Assumed Temperature Exposures for Weather Extremes

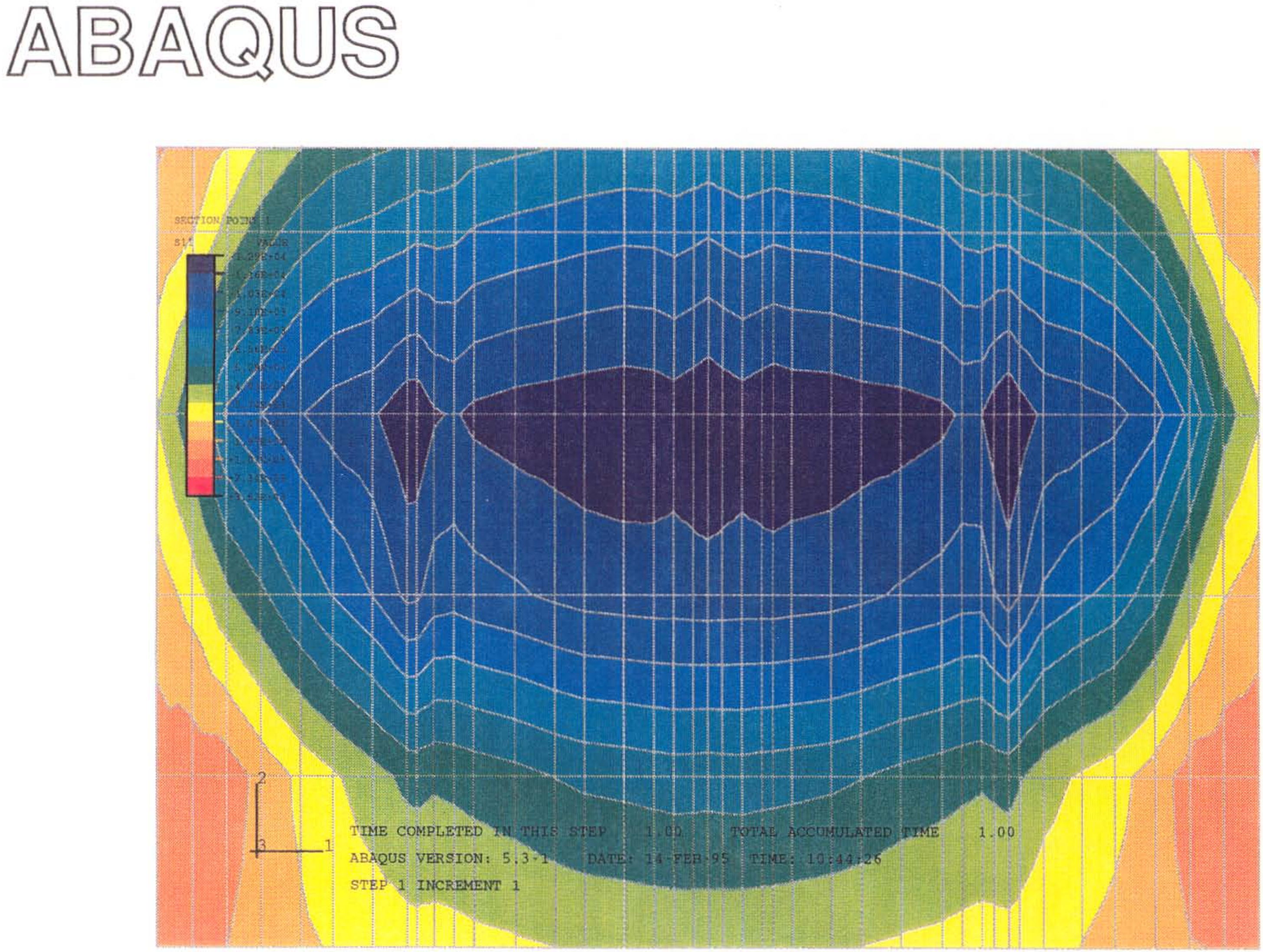

Fig. 29 Longitudinal Thermal Stress Contours for 2-in Wearing Surface (E_p = 760 ksi), Test 5

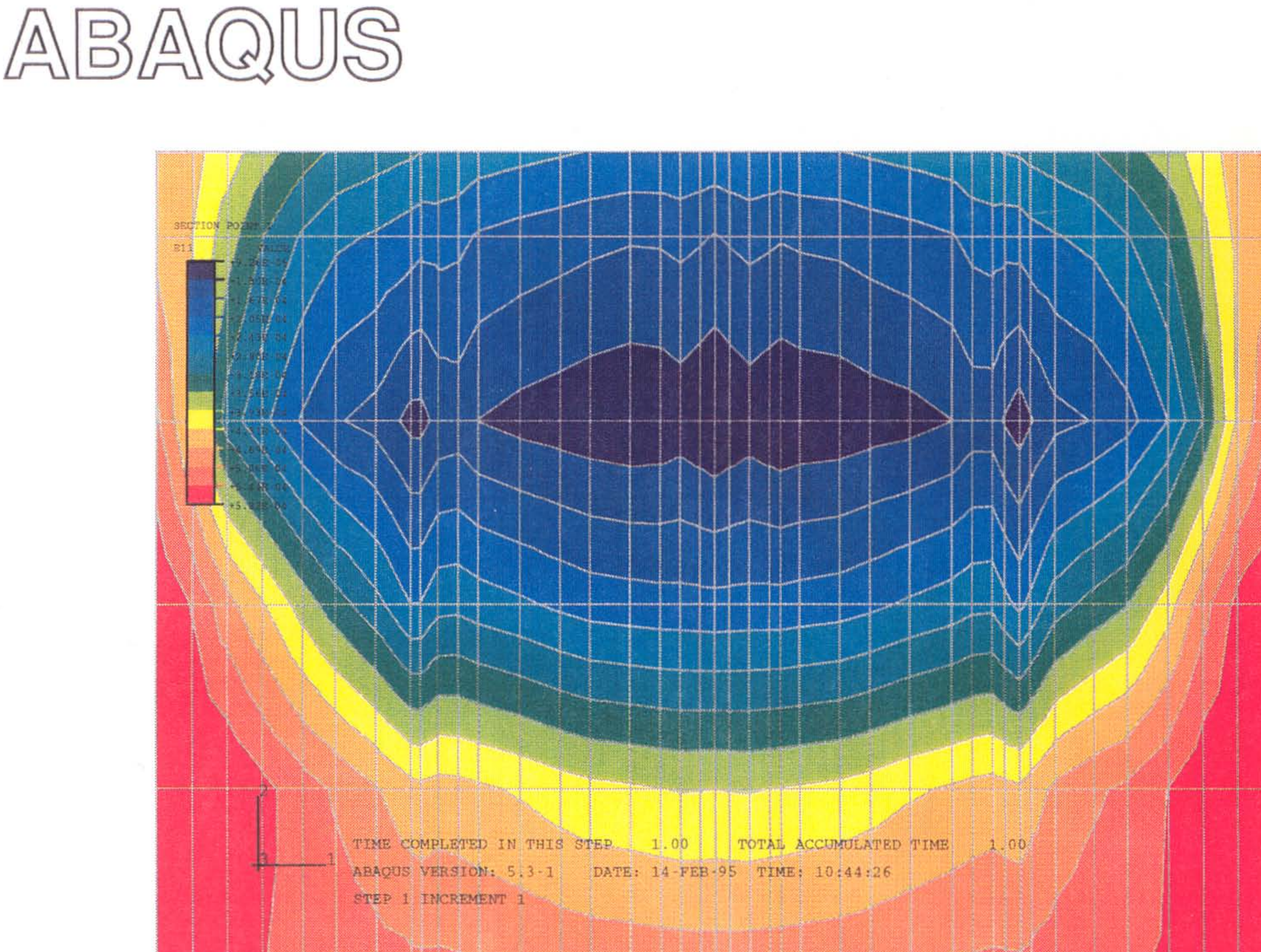

Fig. 30 Longitudinal Thermal Strain Contours for 2-in Wearing Surface (E_p = 760 ksi), Test 5

METHOD FOR SELECTING ALTERNATIVE WEARING SURFACES

A previous section of this report stated that a future wearing surface for the Yukon River bridge should resist abrasion, have good traction properties, resist live load flexural fatigue, and have sufficient ductility to resist cold temperature cracking. A goal of this study was to identify possible wearing surfaces that may perform well under these conditions. These surfaces would be selected for laboratory tests in the next phase. No wearing surfaces were identified by the literature and national survey of Departments of Transportation that suggest that available wearing surface materials will meet the criteria needed for the Yukon River Bridge. Thus, a procedure was developed to assist engineers in selecting a wearing surface for truck loads and expected weather extremes. This procedure would provide a rational method for selecting wearing surfaces for testing traction, abrasion, and adhesion resistance.

ABAQUS was used to calculate two maximums: a) live load tensile strain and the range of live load strain and b) thermal strain and stress for a wearing surface of known thickness for the Yukon River Bridge. Results are presented in Table 32 and 33. Figures 31 and 32 provide the wearing surface live load induced strain and range of strain,and Figures 33 and 34 provide the thermally induced strains and stresses in the wearing surface. These results vary with thickness and material modulus.

If an engineer wishes to select a possible wearing surface for the Yukon River Bridge, Figs 31 through 34, combined with the material properties provided by the supplier, should be sufficient to determine if live load and thermal performance will be satisfactory. The procedure following the figures is suggested for selecting alternate wearing surfaces.

Table 32. Wearing Surface Maximum Tensile Strains for Truck Traffic

Modulus[a] (ksi)	2"[b]		4"[b]		6"[b]	
	Maximum ($\mu\epsilon$)[c]	Range ($\mu\epsilon$)[c]	Maximum ($\mu\epsilon$)[c]	Range ($\mu\epsilon$)[c]	Maximum ($\mu\epsilon$)[c]	Range ($\mu\epsilon$)[c]
350	816	1,814	390	1,043	251	837
760	503	503	217	679	133	475
		-682				
1,500	303	762	134[d]	413	82[d]	283
2,400	207	207	87	295	70	217
		-341				
4,000	138	388	81	225	63	163
5,000	122	209	78	201	59	144

Note: [a]Stiffness of wearing surface
[b]Thickness of wearing surface, inches
[c]Deck strain expressed in micro-strain.
[d]Transverse strains; all others are longitudinal

Table 33. Wearing Surface Thermal Strains & Stresses

Modulus[a] (ksi)	2"[b]		4"[b]		6"[b]	
	Maximum Strain ($\mu\epsilon$)[c]	Maximum Stress (ksi)	Maximum Strain ($\mu\epsilon$)[c]	Maximum Stress (ksi)	Maximum Strain ($\mu\epsilon$)[c]	Maximum Stress (ksi)
350	1,047	7.56	909	4.96	863	4.47
760	915	6.55	821	4.4	778	3.73
1,500	822	5.52	750	3.6	710	2.67
2,400	762	4.68	702	2.8	674	2.26[d]
4,000	721	3.91	642	1.92[d]	627	1.91[d]
5,000	698	3.6	622	1.79	606	1.75[d]

Note: [a]Stiffness of wearing surface
[b]Thickness of wearing surface, inches
[c]Deck strain expressed in micro-strain.
[d]Transverse stresses; all others are longitudinal

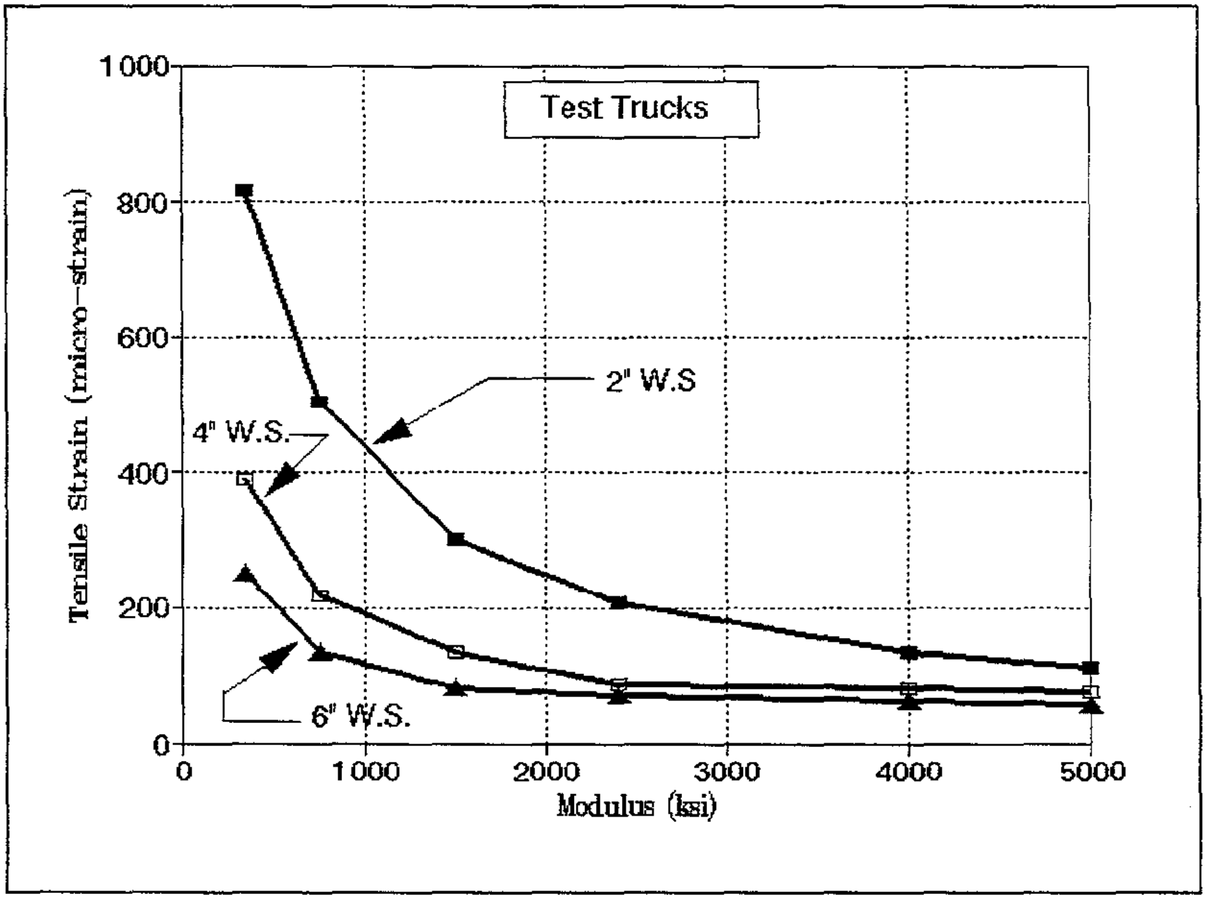

Fig. 31. Wearing Surface Tensile Strain Selection Chart for Static Test Trucks

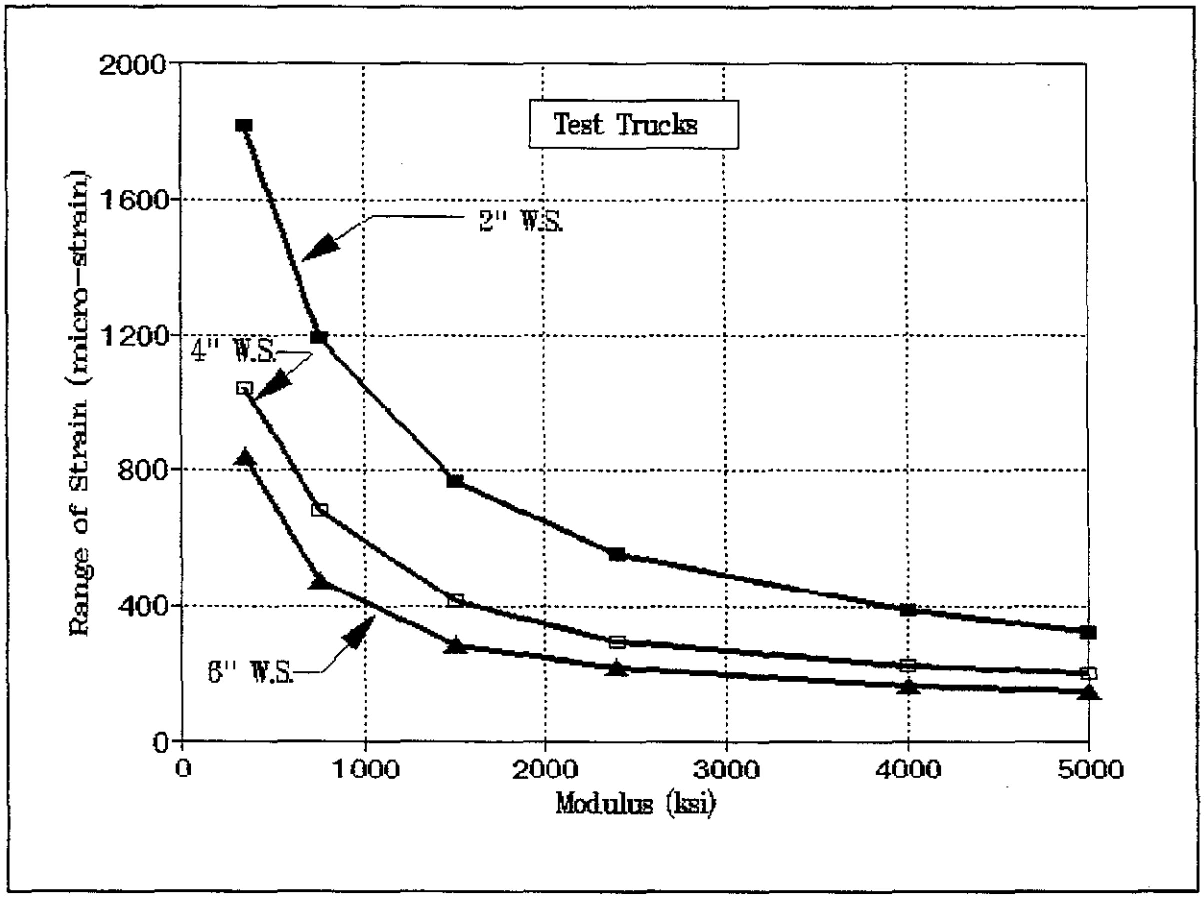

Fig. 32. Range of Deck Tensile Strain Selection Chart for Static Test Trucks

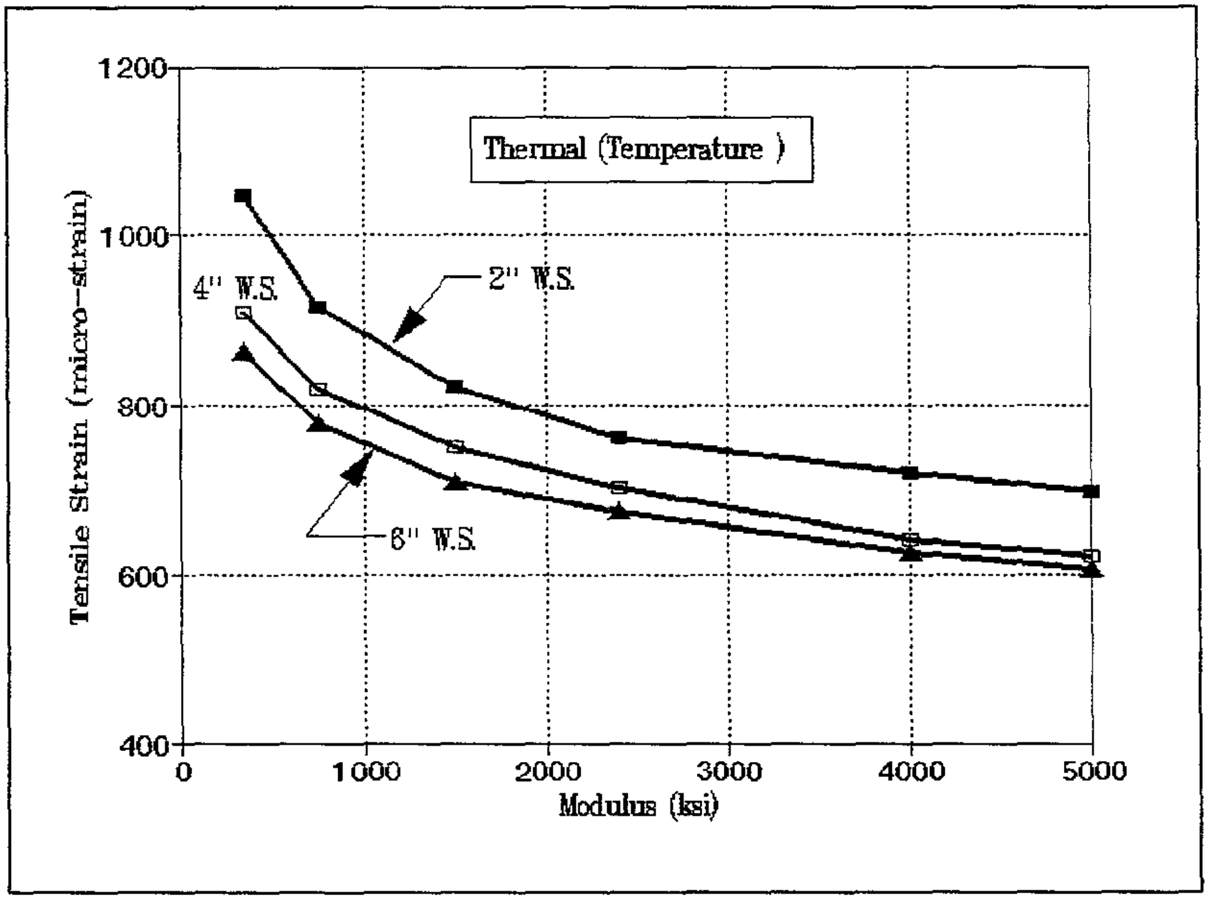

Fig. 33. Wearing Surface Thermal Strains Selection Chart for Temperature Extremes

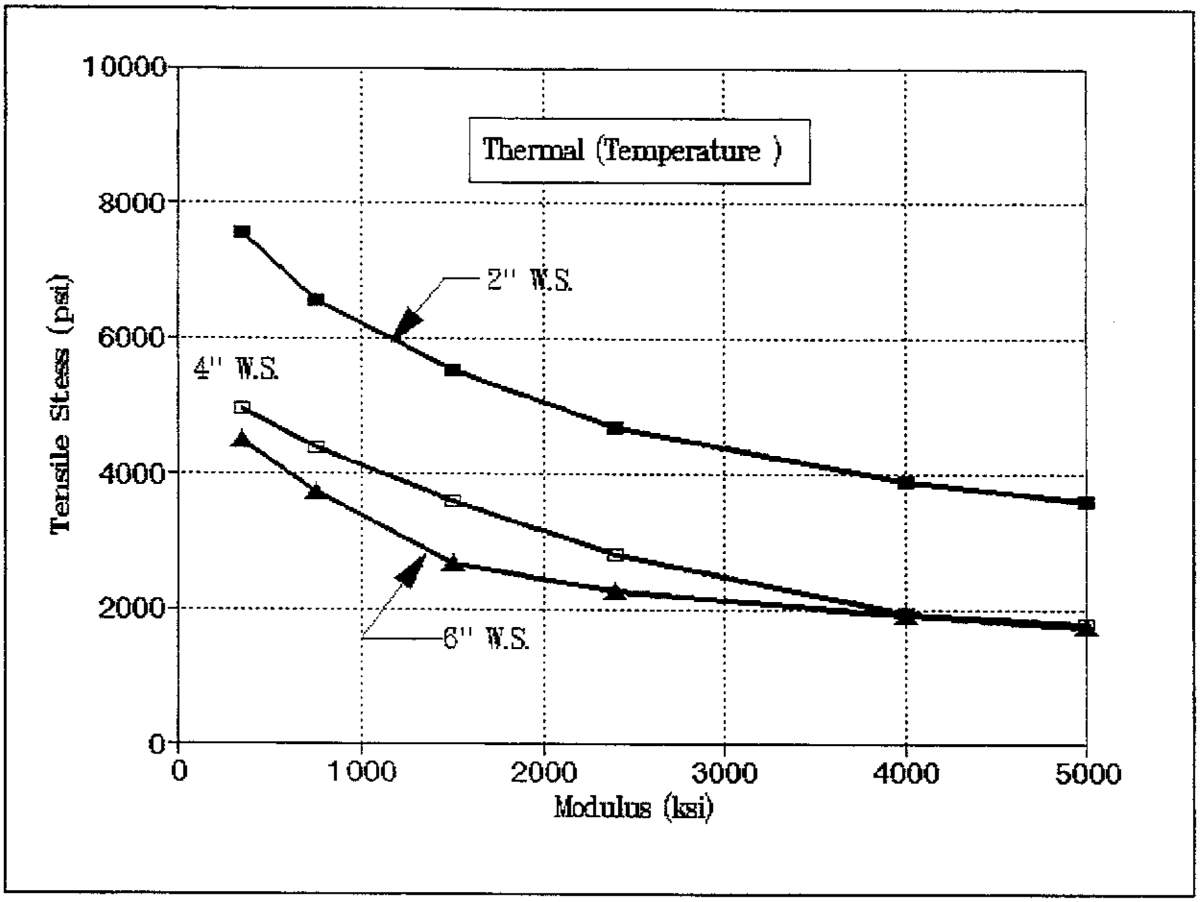

Fig. 34. Wearing Surface Thermal Stress Selection Chart for Temperature Extremes

Step 1. Find the material modulus of a given wearing surface; this information may be obtained from the supplier or from laboratory tests.

Step 2. Obtain the limiting tensile live load strain and the range of strain from the supplier or by testing (for fatigue).

Step 3. Use the material modulus and limiting strain values of steps 1 and 2 to select the thickness by entering Fig. 31 with the maximum tensile strain and material modulus. Then enter Fig. 32 with the range of strain and material modulus and select thickness. The larger thickness between the two charts should prevent flexural live load induced damage to the wearing surface.

Step 4. Obtain from the supplier the limiting cold temperature thermal stresses for the material at $-50°F$. If this information is not available, laboratory tests should be conducted. By knowing the limiting cold temperature thermal stresses for the material selected, the engineer can predict if the material will crack from Figs. 33 and 34.

Step 5. Find the specific weight of the material and thickness selected and calculate stresses for dead load (DL), dead load plus live load (DL+LL) and dead load plus live load plus impact. These stresses should not exceed the stresses allowable in the steel deck, floor beams and steel girder. The additional superimposed weight combined with the live load should not exceed the allowable weights in the substructure.

LABORATORY SPECIMEN SELECTION PROCEDURE

General Comments

Based on the information in the preceeding charts, engineers and suppliers can use the following procedure to select alternative surfaces. Assume several wearing surfaces are to be examined for possible use on the Yukon River Bridge. The factors important to performance for a given wearing surface on this structure should include

- Ductility and possible fatigue resistance of the wearing surface material;
- Thermal crack resistance vs temperature;
- Abrasion resistance vs temperature;
- Surface traction vs grade and temperature; and
- Bond stresses between the steel plate and the wearing surface. Bond stress performance can be expected to vary with shear force and temperature.

Mechanical, fatigue, and thermal properties for each material should be characterized by tests in the laboratory. Additional laboratory tests could be conducted to simulate field behavior of the bridge deck, a wearing surface - orthotropic steel deck composite. For example, are bond and thermal stresses in the wearing surface influenced by the steel deck? What is the stiffness of the wearing surface in relation to the steel deck and how does this influence behavior?

Consider a laboratory test of a simple beam specimen that is composed of a wearing surface bonded to a steel plate. The laboratory beam composite may be sized to account for the stiffness of the wearing surface in relation to the stiffness of the steel deck. If flexural beam strains in the wearing surface material are studied as a function of load cycle, temperature, and shear, assume a flexural test on a simply supported beam with two equal point loads at third points, see Fig. 35. The thickness of both the wearing surface and the steel plate may be determined by principles of mechanics. The procedure will be outline herein. All parameters used in the procedure are shown in Fig. 35.

Yukon River Orthotropic Bridge Deck

Given the concept that a possible wearing surface for the Yukon River Bridge has been selected for study in the laboratory, the modulus of the wearing surface and the steel deck are to be determined. The modulus of the wearing surface is represented by E_p, and the modulus of the steel deck is represented by E_s. The strains for the wearing surface may be determined by Figs. 31 and 32. For purposes of discussion, values in Table 34 will be used to illustrate the procedure for selecting the laboratory test beam geometry.

Table 34. Bridge Parameters for Specimen Selection

Item	Value
Material Modulus:	
Orthotropic steel plate, E_s	29,000 ksi
Wearing surface, E_p [a]	760 ksi
Material Strains:[a]	
Top of wearing surface (comp.), ϵ_{pt}	500 micro-strain
Bottom of wearing surface (tens.), ϵ_{pb}	100 micro-strain
Interface strain (comp.), ϵ_i	107 micro-strain
Bridge Deck Live Load Deflection:	$\delta = 6/100$ [a]
Ratio of live load deflection to bridge span, $\delta = \Delta/S$	

[a] ABAQUS results for a wearing surface with a modulus of $E_p = 760\ ksi$

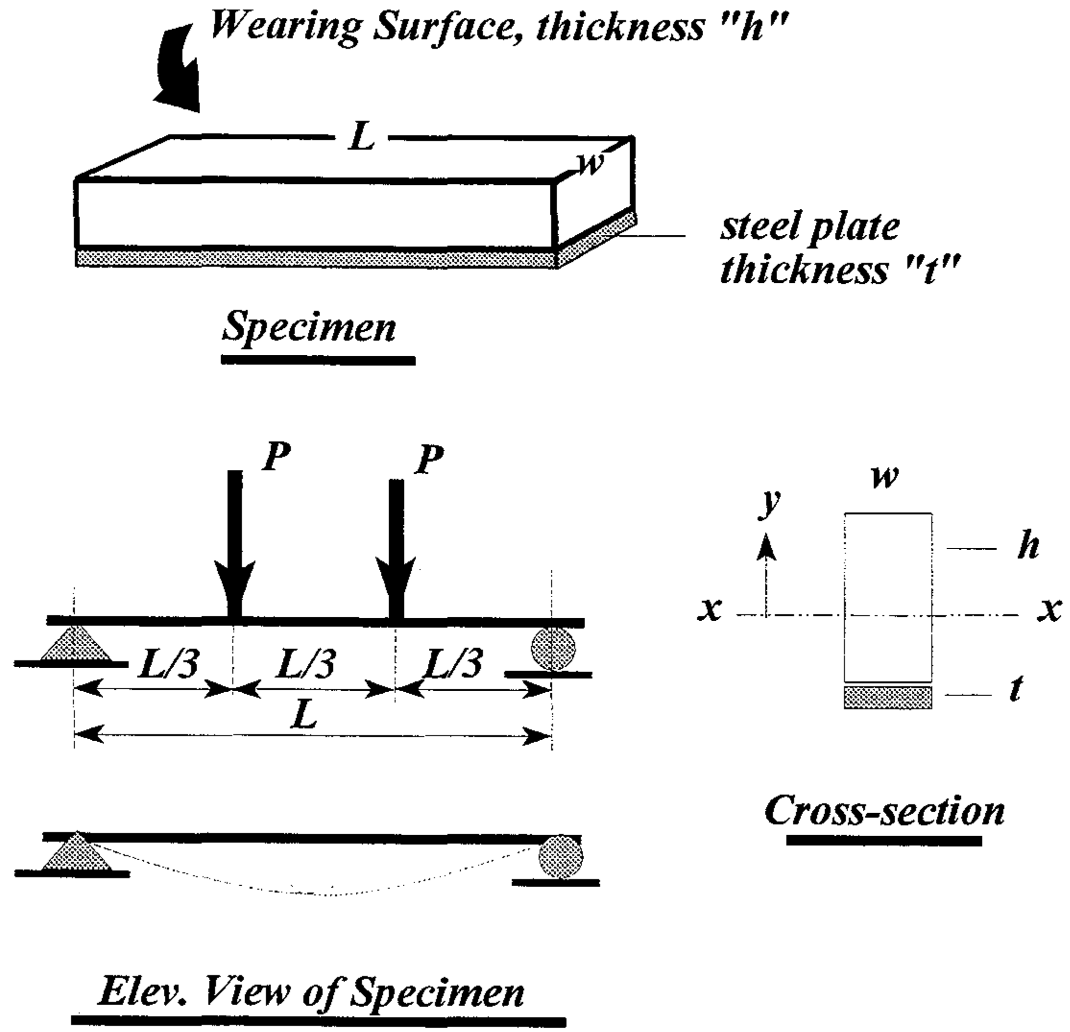

Fig. 35. Recommended Laboratory Test Specimen Geometry

Step 1. Select a wearing surface and corresponding thickness. This should be based on the load conditions and the supplier's recommendation for the limitation of tensile strains, range of strain and conditions for thermal cracking;

Step 2. Determine modulus, E_p, and poisson's ratio, μ_p, for the pavement wearing surface that was selected for study;

Step 3. Select the truck loads to be used for predicting performance of the bridge deck wearing surface. For example, the engineer may choose to study the influence of HS25-44 and other "load permitted trucks";

Step 4. Using the truck loads, peform an analysis to find wearing surface strains. Find the maximum strains at the top, and bottom of the wearing surface. The strain at the wearing surface and steel plate interface should also be determined.

Laboratory Samples

Consider a simply supported beam composed of two materials, see Fig. 35. The purpose herein is to provide a procedure for selecting the geometry for a laboratory beam specimen that will accurately simulate the stiffness of a wearing surface in relation to the stiffness of the orthortropic steel plate on the Yukon River Bridge. First, several dimensionless variables are defined as follows:

$$\begin{aligned} x &= \frac{Pl^2}{E_p I} \\ y &= \bar{y}L \\ z &= \frac{h}{L} \\ u &= \frac{t}{L} \end{aligned} \qquad \qquad (1)$$

in which P is the load to be applied by the testing machine, L is the distance between supports for the simply supported beam, E_p is the modulus of the wearing surface, I is the transformed moment of inertia for the beam, t is the thickness of the steel plate to be used in the laboratory specimen, h is the thickness of the wearing surface to be used for the laboratory beam, and $\bar{y}$ is the centroid of the composite beam.

Anticipated live load strains in the wearing surface and the live load span deflection for the Yukon River Bridge are variables known to be needed to determine size of the laboratory beam specimen. The values needed are the maximum top strain, ϵ_{pt}, and the maximum bottom strain, ϵ_{pb}, and the limiting interface strain, ϵ_i, in the wearing surface and a maximum live load deflection, Δ.

Step 5. Find the modular ratio of the materials in the test beam using the following equation:

$$n = \frac{E_s}{E_p} \qquad \text{(2)}$$

in which E_s is the modulus of the orthotropic steel plate and E_p is the modulus of the bridge deck wearing surface.

Step 6. Find a deflection normalization factor expressed by

$$x = \frac{\delta}{0.0355} \qquad \text{(3)}$$

in which δ is obtained from Table 32. Note, the maximum beam deflection of a simply supported beam to be tested in the laboratory is expressed by

$$\Delta = 0.0355 \frac{PL^3}{E_p I} \qquad \text{(4)}$$

in which P is the concentrated load, L is the length of the beam, E_p is the modulus of the wearing surface, and I is the transformed moment of inertia. The maximum beam moment is given by

$$M = \frac{PL}{3} \qquad \text{(5)}$$

Combining Eqs. 1 and 5, gives the relationship for maximum top and bottom strains and is expressed by

$$\epsilon_t = \frac{M\bar{y}}{E_p I}; \ \epsilon_t = \frac{xy}{3} \qquad \text{(6a)}$$

and

$$\epsilon_b = \frac{M(h+t-\bar{y})}{nE_p I}; \quad \epsilon_b = \frac{x(\alpha+y)}{3} \qquad \text{(7)}$$

Step 7. Find values of z, I, and P. Combining Eqs 1, 6a and 6b gives

$$u^2 + 2(y-\alpha)u + \frac{\alpha}{n-1}(2y-\alpha) = 0 \qquad \text{(8)}$$

Solving Eq 8. gives u_1 and u_2. Values of z_i are obtained by

$$z_i = \alpha - u_i; \ i=1,2 \qquad \text{(9)}$$

The applied transverse load may be calculated by

$$P_i = \frac{E_p I_i x}{L^2}; \quad i=1,2 \qquad \text{..........} \quad \textbf{(10)}$$

Step 8. Find the thickness of the wearing surface, h_i, thickness of the steel plate, t_i, and the sectional centroid. These numbers are found by

$$\begin{aligned} h_i &= z_i L \\ t_i &= u_i L; \quad (i=1,2) \\ \bar{y} &= yL \end{aligned} \qquad \text{..........} \quad \textbf{(11)}$$

Step 9. Based on the desired interface strain, ϵ_i, and the centroidal distance, $\bar{y}$, t and h can be adjusted for a given w and L. This adjustment provides a procedure for the investigator to select the appropriate geometry to approximate the influence of the steel deck stiffness on the test strain result for a given wearing surface.

SUMMARY AND CONCLUSIONS

Live load strains in future wearing surfaces on the Yukon River Bridge should be low. The magnitude of strain varies with load, truck geometry, thickness of the wearing surface and modulus of the wearing surface. The range of strain for a live load is also expected to be low. The expected thermal stresses will be high.

The authors found several wearing surfaces for orthotropic steel bridge decks that have satisfactory performance. These wearing surfaces were found in the literature and through a national survey of departments of transportation. Since the wearing surfaces found through this research are subjected to conditions different than the Yukon River bridge, they may not perform well for this structure. For example, all other bridges experience large traffic volumes, moderate temperatures and a relatively flat grade. The Yukon River bridge carries a small amount of traffic, carries heavy trucks, is subjected to extreme winter temperatures (-50° F) and the grade is steep (about 6%). The better performing wearing surfaces on other orthotrpic bridge decks include Transpo T-48, epoxy asphalt over a coal tar epoxy, stone mastic asphalt (SMA), asphalt rubber stress absorbing membrane interlayer (SAMI), polymer modified concrete with steel fibers overlayed with asphaltic rubber, polymer modified asphaltic concrete, and polyurathean impregnated with stone chips.

A method was developed and presented for selecting a wearing surface to resist flexural fatigue and thermal cracking. An example for using this method is presented.

A wearing surface for this structure will be expensive. Therefore, it is suggested that several promising wearing surfaces be selected for study. Abrasion resistance, thermal cracking, traction, and bonding resistance during cold temperatures will be important to the performance of a given wearing surface.

RECOMMENDATIONS FOR FUTURE RESEARCH

The results of this study show that live load strains and the range of strains for a future wearing surface are expected to be low. Thermal stresses in a wearing surface will be large. These results suggest that material fatigue caused by truck loads will be unlikely. However, thermal cracking may occur in most wearing surface materials during winter low temperatures. The results from the national survey and the literature on wearing surfaces for othrothopic bridge decks were insufficient in finding a structure that had conditions similar to the Yukon River Bridge.

Therefore, it is suggested that the remaining two phases be implemented to provide sufficient data to help address the following needs:

- Sufficient ductility to accommodate, without cracking or delamination, any expansion or contraction of the steel plate;
- Sufficient fatigue strength to withstand flexural cracking due to deck plate deflections;
- Sufficient durability to resist rutting, shoving and wearing;
- Sufficient surface protection to remain impervious to water, motor fuels and oils; and
- Sufficient surface protection to resist deterioration from de-icing chemicals and petroleum distillates.

Experiences with orthotropic deck wearing surfaces by other state agencies are of limited value for the following reasons:

- The Yukon River orthotropic steel deck bridge may have steeper grades (6+ %) than is common in other areas.

- Conditions imposed by extreme temperatures, heavy truck loads, and low volumes of traffic may not allow extrapolation of results obtained by others.

- The deck/bridge flexibilities for the Yukon River bridge may be different from that found for other areas.

- The amount of snow plowing and/or the use of deicing chemicals may be different for the Yukon River bridge.

As a reminder to the reader, the next phases that were originally planned are as follows:

Phase 2 (next study). Using the information from Phase 1, surfacing alternatives would be selected for study. Prior to study, analytical interface strains predicted for the loads and conditions at the Yukon River Bridge would be calculated using a computer model. Laboratory material tests would then be conducted to simulate field strains and loading conditions. It would be the objective of Phase 2 to evaluate, through laboratory studies, the performance of possible surfacing alternatives for the bridge deck.

Phase 3 (third study, field application). During this phase, researchers would instrument and monitor the performance of field sections at the Yukon River Bridge. The objective of this phase is to provide bridge engineers with performance data for experimental surfacing alternatives. The results from this phase should provide bridge engineers with data that will improve economical long term decisions.

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APPENDIX

Letter and Questionaire for Survey of DOT's in the "Lower 48"

``DATA mis.doc''

September 20, 1993

``name''
``dept''
``address''
``city''

Attention: Bridge Design

``SA''

The Alaska Department of Transportation and Public Facilities (AKDOT&PF) has a 6 span 2,295 ft orthotropic steel bridge that crosses the Yukon River on a 6% grade about 100 miles north of Fairbanks, Alaska. The bridge was designed to transport traffic, an oil pipeline, and 2 future natural gas lines. The bridge was built in 1976 to provide transportation for the Alaska pipeline expansion.

In 1979, a laminated timber wearing surface was installed over the orthotropic steel deck. This wearing surface was to be a temporary solution but the timber surfacing was replaced with a similar solution in 1992.

Problem:

Normally, truckers use chains to cross this bridge during the winter. The timbers are planned by tire chains causing traction to be reduced with time. Snow and ice builds up on the wearing surface, temperatures drop below -60°F, and there appears to be inadequate protection to the steel deck, the life of the wearing surface is only 15 years and the cost of replacing the wearing surface is significant.

Therefore, the Alaska Department of Transportation and Public Facilities is looking for alternative materials that can be used as a wearing surfacing. These materials should: a) provide protection to the steel deck; b) maintain sufficient traction for a 6% grade; c) be ductile in extreme cold temperatures; d) have a longer life; and e) be less expensive.

AKDOT&PF requested the University of Alaska Fairbanks to compile alternative solutions that have been used successfully on orthotropic steel bridge decks. Towards this end, the authors prepared a brief questionnaire to seek answers to questions that are believed important to answering this question. The questionnaire consists of two parts. Part 1 will help us evaluate experiences by DOT's with wearing surfaces over orthotropic bridge decks. Part 2 is devoted assembling the names of software that is used by DOT's to analyze orthotropic steel bridge decks. It will be appreciated that we received your response by October 30, 1993.

Please send your response to:

J. Leroy Hulsey, Associate Professor
Department of Civil Engineering
248 Duckering Building
University of Alaska Fairbanks
Fairbanks, Ak. 99775
Phone: (907) 474-7816
FAX: (907) 474-6807

We thank you in advance for your time and assistance with this important data.

sincerely,

Kevin Curtis
J.Leroy Hulsey
Lutfi Raad

BRIDGE DESIGN QUESTIONNAIRE

Return to: J. Leroy Hulsey, Associate Professor
Department of Civil Engineering
248 Duckering Building
University of Alaska Fairbanks
Fairbanks, Ak. 99775
(907) 474-7816
(907) 474-6807

1. Please indicate your state DOT ________________________(state)

2. Does your state have any bridges with orthotropic steel decks ________________________(yes,no)

Part. 1. Orthotropic Bridge Deck Experiences?

1). How many orthotropic bridge deck bridges does your state currently have/or maintain? ____________________________(give number).

2). Do you have any long span (> 300 ft) orthotropic steel bridge decks?
________ (yes/no)
________ (if yes, number exposed to freezing temperatures)

3). If the answer to Question 2 is yes?

a) Do any of these bridges have a wearing surface on the orthotropic steel plate ? ________ (yes,no)

b) What type of wearing surface does your state use over orthotropic steel bridge decks? ____(name the system(s))

4). In your state, what type of wearing surfaces have you found to perform the best (economical with less maintenance) for orthotropic steel bridge decks?

a) . . . . ___________________________________ (name the material(s))
. . . . ___________________________________ (describe the system)

b) . . . . ___________________________________ (name the material(s))
. . . . ___________________________________ (describe the system)

5). Has your state found an attachment system that has successfully bonded wearing surfaces to steel decks?
____________ (yes/no); ____(if yes, describe the sytem)

6). What is the expected life of your best wearing surface that is used over orthotropic steel bridge decks?

________________________________ type ; _______(years).

7). Are the wearing surface materials described in Question 4 exposed to below freezing temperatures during the winter? . . . . __________ (yes, no)

8). Have you found a bridge deck wearing surface that will provide traction on steep grades, i.e. about a 6% grade?

__________yes/no; ________________________________ (if yes, name the product).

9). Has your state tested the use of wearing surfaces on steel orthotropic decks?

__________ yes/no; ________________________________(if yes, name the research report).

10). If you have a wearing surface material that you have used successfully for other applications and think it may work for the conditions described in this survey, please provide the name and description of the material

Name: ____

Description: ____

____

____

Part 2. Software Questions:

11). What is the name of the software you use (would use) to analyze/design orthotropic steel decks for highway loads? That is, what programs are used to calculate stresses in the steel plates and stringers?

__________________________ (give the program name).

a). What type of analysis method(s) is (are) used by these programs?

________________________________(finite element, finite difference, other).

b). If a software name is provided, please provide the contact individual that supplies the software?

Name: . . . . ______________________________
Company: . . . . ______________________________
City: . . . . ______________________________
State: . . . . ________________ zip: ________
Phone: . . . . ______________________________

12). May Universities or other DOT's obtain this software?

______________________ (yes, no, don't know)

13). Is the software easy to use? ________________________________(yes, no)

14). In your opinion, does the software provide reliable results? _______________(yes, no)