SYNTHESIS INTO THE CAUSES OF CONCRETE BRIDGE DECK CRACKING AND OBSERVATIONS ON THE INITIAL USE OF HIGH PERFORMANCE CONCRETE IN THE US 95 BRIDGE OVER THE SOUTH FORK OF THE PALOUSE RIVER

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16. Abstract

In recent years, the Bridge Section of the Idaho Transportation Department (ITD) has observed increased cracking in concrete bridge decks. This increase in cracking has occurred in concrete bridge decks that have used designs that historically produced satisfactory performance. The cracking has occurred both in new decks and in decks that have been subjected to traffic for various periods of time.

To determine possible reasons for the increased cracking, researchers at the University of Idaho have performed a literature review of articles, papers, and standards focused on bridge deck cracking. In addition, the deck of a bridge for highway US 95 constructed over the South Fork of the Palouse River was instrumented with strain and temperature gauges. This project was the first in the State of Idaho involving the use of HPC for the bridge deck. The weather and concrete placement procedures were also monitored and material testing was performed on the deck concrete. The deck was placed in two stages; the first stage portion of the deck was constructed using the conventional Idaho Class 40A mix, while the second stage portion was constructed using a high performance concrete mix.

The results of the monitoring and testing on both stages of the bridge deck were then compared to the literature review to determine the cause of cracking in the deck. In addition, the report compares the concrete used in the two bridge decks to determine if the high performance concrete mix provided any improvement with respect to cracking. Finally, the report presents recommendations on how to reduce cracking. Results from the monitoring and testing of the stage 1 deck indicated that cracking in the concrete was mostly due to restraint of the deck by the girders and parapet wall. Uplift from skew and high heat of hydration temperatures were the main causes of tensile stress build up in the deck, compounded by the low creep and high modulus of elasticity of the concrete used. Results from the monitoring and testing of stage 2 indicated that cracking in the concrete was also mostly due to restraint of the deck by the girders and parapet wall.

Reducing the cement content, adding fly ash to the mix, decreasing skew, and/or reducing deck restraint appear to be effective in reducing deck cracking.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

ABSTRACT

In recent years, the Bridge Section of the Idaho Transportation Department (ITD) has observed increased cracking in concrete bridge decks. This increase in cracking has occurred in concrete bridge decks that have used designs that historically produced satisfactory performance. The cracking has occurred both in new decks and in decks that have been subjected to traffic for various periods of time.

To determine possible reasons for the increased cracking, researchers at the University of Idaho (UI) have performed a literature review of articles, papers, and standards focused on bridge deck cracking. In addition, the deck of a bridge for highway US 95 constructed over the South Fork of the Palouse River was instrumented with strain and temperature gauges. This project was the first in the State of Idaho involving the use of HPC for the bridge deck. The weather and concrete placement procedures were also monitored and material testing was performed on the deck concrete. The deck was placed in two stages; the first stage portion of the deck was constructed using the conventional Idaho Class 40A mix, while the second stage portion was constructed using a high performance concrete mix.

The results of the monitoring and testing on both stages of the bridge deck were then compared to the literature review to determine the cause of cracking in the deck. In addition, the report compares the concrete used in the two bridge decks to determine if the high performance concrete mix provided any improvement with respect to cracking. Finally, the report presents recommendations on how to reduce cracking.

Results from the monitoring and testing of the Stage 1 deck indicated that cracking in the concrete was mostly due to restraint of the deck by the girders and parapet wall. Uplift from skew and high heat of hydration temperatures were the main causes of tensile stress build up in the deck, compounded by the low creep and high modulus of elasticity of the concrete used. Results from the monitoring and testing of Stage 2 indicated that cracking in the concrete was also mostly due to restraint of the deck by the girders and parapet wall.

Reducing the cement content, adding fly ash to the mix, decreasing skew, and/or reducing deck restraint appear to be effective in reducing deck cracking.

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BACKGROUND INFORMATION

Strains in a concrete bridge deck are caused by three main mechanisms: 1) external loading from traffic and dead loads 2) thermal changes and 3) shrinkage of the material matrix. The deck is easily designed for the external loading strains and concurrent stresses; however, it must also undergo the undesirable yet inevitable strains caused by thermal changes and shrinkage. As long as the concrete is not physically restrained, the temperature and shrinkage changes will not develop any stresses in the bridge deck; however, decks are usually constrained and tensile stresses from these changes do occur. When the longitudinal stresses exceed the modulus of rupture of the concrete, transverse cracks form. Although the cracks do not have a significant effect on the structural stability of the bridge, they can allow corrosive chemicals and water to reach the reinforcing bars, thereby accelerating the deterioration of both the concrete and the reinforcing bars, and ultimately reducing the service life of the deck. As the deck ages, the transverse cracks become more severe. Alkali-silica reaction (ASR) effects tend to cause random map cracking, rather than regularly spaced transverse cracks. Cracking due to ASR tends to occur in areas of high moisture content, such as piers, retaining walls, and other foundation elements.

Section 1.1 discusses the internal factors behind the causes of transverse cracking in the bridge deck. Most of the causes that are discussed had little to no effect on cracking in the case study bridge deck for this project; however, drying shrinkage and thermal changes did cause tensile strains and thus stresses that eventually caused transverse cracking in the bridge deck. The main cause of the tensile strains was from the large thermal changes experienced by the bridge deck due to high cement hydration temperatures. Although not evident at the time this paper was written, traffic induced vibrations and repeated deflections may increase the sizes of the existing cracks on the case study bridge deck.

Section 1.2 discusses the external factors influencing the causes of transverse cracking discussed in Section 1.1. The factors include: environmental conditions, design details, material properties, and construction procedures. For this project, the environmental conditions and construction procedures were mainly affected by the air temperature, which was around 40° F at the time of concrete placement. The main influencing factors for transverse cracking in the case study bridge deck were from design details and material properties. More specifically, the end fixity, girder type, girder size, girder spacing, and reinforcement alignment causing increased restraint on the deck, discussed in the design details section, and cement content and lack of fly ash increasing hydration temperatures, discussed in the material properties section. Recommendations from the literature to decrease the risk of transverse cracking are then presented in Section 1.3.

1.1 Causes of Transverse Cracking in Concrete Bridge Decks

Transverse cracking can be caused by several different reasons. Sections 1.1.1 through 1.1.8 present a summary of each reason based on the literature review

1.1.1 Plastic Shrinkage

Immediately after concrete is placed, the heavier aggregates tend to settle and the free water in the matrix rises to the surface of the deck in a process known as bleeding. If the evaporation rate exceeds the rate of the bleed water rising from the full depth of the deck while the concrete is still in a plastic state, then water needed for proper hydration of the cement begins to evaporate from the surface and plastic shrinkage occurs. The subsequent cracks that form from this shrinkage are usually shallow, 2 or 3 inches deep, and usually no longer than 2 or 3 feet(1). Evaporation from the surface increases as concrete temperature, air temperature, and wind speed increase and the relative humidity decreases. As long as the evaporation rate, determined using Figure 1-1, does not exceed the rate of bleed water rise, approximately equal to 0.2 lb/ft^2/hr for normal water-to-cement ratio concrete and 0.1 lb/ft^2/hr for water-to-cement (w/c) ratios less than 0.40, construction precautionary measures to reduce evaporation are not required because plastic shrinkage will not occur(1). For this project, plastic shrinkage did not appear to be the cause of cracking.

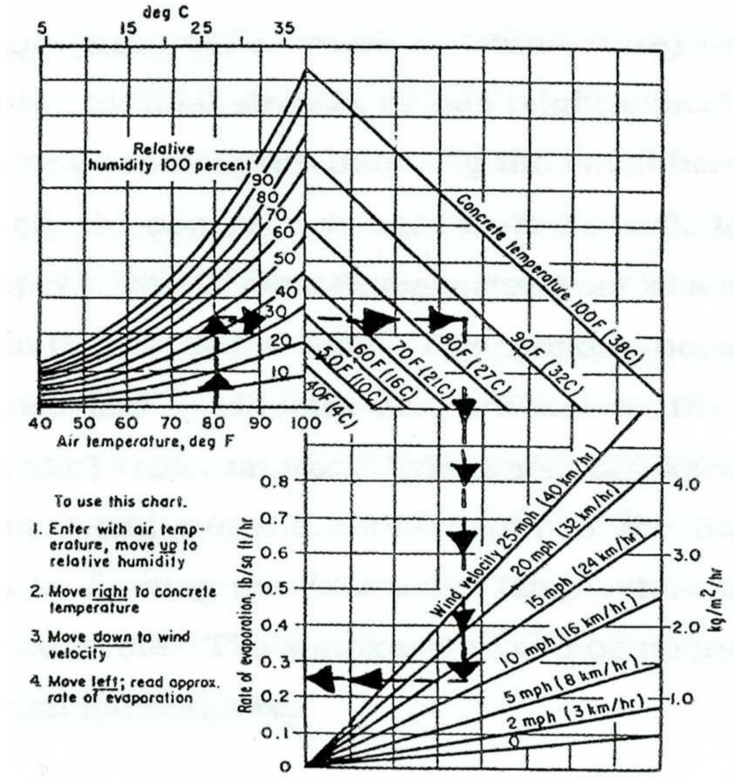

Figure A-1: Rate of Evaporation Above a Freshly Poured Deck ***(2)***.

1.1.2 Drying Shrinkage

If the water in the concrete mix is not evaporated out as bleed water or used in hydration of cement paste to form C-S-H (Calcium Silicate Hydrate), it is known as absorbed water. Absorbed water is a layer of water molecules loosely bonded to the C-S-H particles, holding them apart[3]. When the water molecules evaporate during and after curing, it causes C-S-H particles to come closer together, resulting in drying shrinkage. The higher the water content in the matrix, the farther the water molecules hold the C-S-H particles apart and the more drying shrinkage that will occur. This shrinkage usually causes strains of 400 to 1400 microstrain depending on the volume-to-surface area ratio of the deck, concrete mix ingredients, environmental conditions, and physical restraint of the deck[1,3]. Full-depth drying shrinkage cracks typically begin to form at a restrained 400 microstrain and usually develop above the uppermost transverse bars[1]. Since the drying occurs over a period of time, creep acts beneficially to relieve the stress build up caused by drying and reduce drying shrinkage cracks. A Minnesota study suggests that the rate of shrinkage, not the ultimate shrinkage, has more of an affect on the amount of drying shrinkage cracks that develop[4]. Although drying shrinkage was a cause of tensile strains for this project, it was not the dominate cause of the tensile strains or transverse cracking.

1.1.3 Autogenous Shrinkage

When cement consumes water for hydration purposes, it takes up less space than the cement and water particles separately before hydration, causing autogenous (self-generated/chemical) shrinkage. As long as the water-to-cement ratio is greater than about 0.42, drying shrinkage is the dominant volume change and autogenous shrinkage only represents about 5 percent of the total shrinkage[5, 6]. However, as the water water-to-cement ratio decreases, the autogenous shrinkage increases and can reach 50 to 400 microstrain, as much as half of the total shrinkage for water-to-cement ratios of 0.30[5]. Autogenous shrinkage is especially detrimental to concrete because it occurs during the first several days of hydration when the concrete is still in a plastic or low strength state. By delaying the time in which the initial hydration occurs, by adding retarders or pouring during cold weather, autogenous shrinkage has more time to occur and can become increasingly severe[5]. Autogenous shrinkage can be minimized by avoiding extremely low water-to-cement ratios (below 0.40) and high paste volumes[5]. For this project, autogenous shrinkage was minimal compared to drying shrinkage and had minimal effect on transverse cracking.

1.1.4 Thermal Changes

Hydration in the cement paste causes the temperature of the concrete to rise initially and it usually peaks within 24 hours[1]. As expected, the increased temperature causes expansion of the deck; however, while the concrete is in its plastic-to-hard state, the modulus of elasticity for the deck concrete is not high enough to cause development of compressive stresses. The girders, steel or concrete, that support the deck usually have enough mass that they dissipate the deck’s heat of hydration temperatures and maintain their temperature close to ambient air temperature and their length remains the same as when the deck was poured. When the hydration of the concrete has slowed down and the deck begins to cool, the girders restrain the subsequent shrinkage of the deck. Since the modulus of elasticity for the deck increases as the deck concrete hardens, tensile stresses develop in the top of the deck, which will be in addition to any shrinkage stresses that also develop. Although cracks do not usually form until a restrained microstrain of 230, it is recommended that the restrained microstrain from thermal changes be kept below 150. This can be accomplished by limiting the temperature difference between the deck and the girders to no more than 22° F for the first 24 hours[1]. For this project, thermal changes were the main cause of tensile strains and thus transverse cracking.

1.1.5 Settlement

Before bleeding stops after the concrete is placed, the fresh concrete tries to settle. Horizontal reinforcement resists the settlement, causing cracks to form above and parallel to the uppermost reinforcing bars. The vertical plane of weakness from the resisted settlement and the subsequent crack that is formed is shown in Illustration 1-1. Settlement cracks decrease with decreasing slump and bar size and increase with increasing clear cover. Concrete with water-reducing admixtures may not show the same trend in slump. Table 1-1 shows the probability of settlement cracking based on these three variables and shows that settlement cracking has the smallest probability of occurring (~0 percent) when there are 2 in. of clear cover, 2 in. slump, and No. 4 rebar as reinforcement. For this project, settlement cracking did not occur.

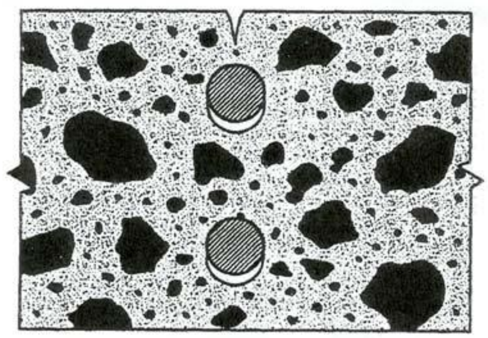

Illustration 1-1: Subsidence Cracking[1].

Table 1-1: Probability of Settlement Cracking[7].

Bar Size	Probability of Cracking Predicted by Regression Analysis (%)								
	2 in. Slump			3 in. Slump			4 in. Slump		
	No. 4	No. 5	No. 6	No. 4	No. 5	No. 6	No. 4	No. 5	No. 6
¾ in. Cover	80.4	87.8	92.5	91.9	98.7	100.0	100.0	100.0	100.0
1 in. Cover	60.0	71.0	78.1	73.0	83.4	89.9	85.2	94.7	100.0
1½ in. Cover	18.6	34.5	45.6	31.1	47.7	58.9	44.2	61.1	72.0
2 in. Cover	0.0	1.8	14.1	4.9	12.7	26.3	5.1	24.7	39.0

1.1.6 Flexure in Plastic Concrete

When pouring continuous span structures without shoring, tensile strains can develop in the top of the deck due to the negative moments over interior supports. Since the modulus of rupture is relatively low while the concrete is setting, transverse cracks can form over transverse reinforcement. Typically the strains related to this type of cracking are related to curvature by the following equation from bending theory:

Equation 1-1: Radius of Curvature

$$\rho = \frac{\varepsilon}{Y},$$

where ρ, ε, and *Y* represent curvature, extreme fiber strain, and the distance from the neutral axis to the extreme fiber, respectively. Values for allowable curvature (before cracking occurs) are $5x10^{-4}$/in and $4x10^{-4}$/in for a deck thickness of 6 in. and 7.5 in. respectively[8]. Proper casting sequence (pour center span first) could decrease the amount of transverse cracking. For this project, flexure in plastic concrete did not appear to cause any cracking.

1.1.7 Flexure under Service Condition

Over interior supports, negative moments are created from dead and live loads. Design can minimize crack widths (decreasing the deterioration potential) by distributing the deck reinforcement in tension zones, decreasing stresses present in the reinforcement steel, and decreasing the cover. Maximum crack widths to be expected can be found by the following equation:

Equation 1-2: Crack Width Estimation

$$Z = F_s \cdot \sqrt[3]{d_c * A}, \text{ where } Z = \frac{W}{0.091},$$

where *W* symbolizes the crack width at the surface (thousandth of inches), *Fs* symbolizes the tensile stress in the steel at the load at which the crack width is to be determined (ksi), *dc* symbolizes the thickness of the concrete cover from extreme fiber to center of closest rebar, and *A* symbolizes the effective concrete tension area[8]. Paying attention to the construction sequence during unshored construction can decrease transverse cracks created by negative flexure of dead loads. This can be achieved by pouring the center portions of adjacent spans before pouring over the interior support as suggested to limit plastic curvature cracking.

For this project, flexure under service conditions did not cause transverse cracking on the top of the deck.

1.1.8 Repeated Deflection and Traffic-Induced Vibrations

Traffic live loads create deflection reversals in decks as vehicles move on and off the bridge. Live load deflection consists of two components: 1) static deflection and 2) dynamic deflection (vibration). Static deflection is defined as "the deflection that would occur if the speed of the vehicle is close to zero" and dynamic deflection is defined as "the deflection resulting from disturbances in the vehicle caused by speed while passing over irregularities on the deck surface[(8)]. Deflection reversals play a larger role in the widening and deepening of cracks rather than actually starting them. Bridge decks usually crack before traffic loads are applied and thus traffic-induced vibrations and vibration frequency have been found to have little effect on the initiation of transverse cracking[(9).] For this project, repeated deflections and traffic-induced vibrations may increase crack widths overtime.

1.2 Factors Influencing Transverse Cracking

The causes of transverse cracking discussed in Section 1.1 are influenced by a complex interplay of a multitude of factors. Although one factor may influence the amount of cracking more than another, it is rarely the only factor that is causing cracking. Sections 1.2.1 through 1.2.4 discuss the various factors that influence transverse cracking.

1.2.1 Environmental Conditions

Environmental conditions such as air temperature, relative humidity, solar radiation, wind speed, and precipitation can greatly influence how a concrete deck performs. They not only affect the deck as it is being poured by changing the amount of evaporation and the initial hydration temperature, but it also affects the long term thermal stresses caused by the daily (diurnal) and yearly temperature changes.

During the initial curing of the deck while the concrete is still plastic, the evaporation rate and the ambient air temperature have the greatest influence on the deck performance. As discussed in Section 1.1.1, evaporation rate increases with an increase in air temperature and wind speed and decreases with an increase in relative humidity. It also increases with an increase in concrete temperature because the concrete heats the air directly above the deck and reduces the relative humidity. The evaporation rate has the greatest affect on short-term crack growth; however, it can increase the amount and severity of long-term crack growth by increasing drying shrinkage. Ambient air temperature has a greater influence on long-term crack growth because of its affect on hydration temperature.

During the initial hydration period, the concrete in the deck expands because of the increase in temperature from hydration. Since the modulus of the deck concrete is relatively low and the girders stay at ambient air temperatures, the girders remain at ambient conditions. When hydration has slowed and the concrete deck cools, it shrinks and tensile stresses develop in the top of the deck because of the restraint from the girders. The higher the peak hydration temperature is; the higher the stresses that can develop in the top of the deck. Therefore, it is extremely important to limit the peak hydration temperature by limiting the air temperature when the deck has reached its peak temperature.

It is also important to reduce the rate at which the deck cools from peak hydration temperature. This can be achieved by pouring the deck in the early morning, mid-evening, or late evening and insulating the deck[(9)]. If the girders are steel, it is equally important to make sure the ambient air temperature is not too low when pouring due to the tensile stresses that develop in the deck when the girders expand from the temperature increase when air temperatures warm up. Therefore, the ideal temperature for pouring a deck is between a high of 65° F to 70° F and a low of 45° F to 50° F. However, as long as the ambient air temperature does not go outside the range of 40° F to 85° F, the risk of cracking is reduced[(4)].

After the concrete hardens and the deck goes into service, thermal stresses develop in the deck due to diurnal and yearly temperature changes. Usually the yearly temperature changes have little effect on the deck, especially when supported by concrete girders, because the deck has a uniform temperature change throughout and the length change of the girders is approximately the same amount as the length change of the deck due to similar thermal expansion rates[(9)]. However, diurnal temperature changes cause nearly linear temperature gradients in the bridge deck, which causes curvature in the deck. In the morning, solar radiation heats the top of the deck faster than the ambient air temperature and conduction heats the bottom of the deck and the girders, creating tensile stresses in the top of the deck from restrained convex upward curvature. Conversely, the top of the deck radiates heat and cools faster in the evening or when it rains, causing a linear temperature gradient and tensile stresses in the bottom of the deck[(9)]. If the tensile stresses exceed the modulus of rupture, transverse cracks will form. Typically, the diurnal temperature cycles in the deck are larger than the ambient air temperature cycles because solar radiation adds energy into the deck[(9)].

1.2.2 Design Specifications

The bridge geometry, concrete deck design, and girder type and design can all have a significant influence on the amount and severity of transverse deck cracking. The most significant design factor is the restraint of the concrete deck. If the deck were not composite with the girders, only the reinforcement in the deck would restrain the strains that develop from thermal changes, shrinkage, and flexure. The embedded reinforcement provides almost no restraint compared to the girders. If there are no strains, there are no stresses and thus no transverse cracks would develop. However, non-composite design of girders is not economical and design constraints usually force the designer to use shear studs to make the deck composite with the girders. Assuming composite design, the end fixity of the girders is another design item that can cause additional restraint in the deck. When the ends of the girders are cast integrally with the abutments, which is often the case for precast girders, the girder supports act like fixed connections for lower level forces such as those imposed from shrinkage, thermal changes, and lower service loads. Since the girders are not allowed to shrink as much as they would be able to for simple supports, they provide even more restraint on the deck, increasing the percentage of transverse cracks(10).

After restraint, the span type, deck thickness, alignment of the top and bottom reinforcement, girder type, and girder size are the next most influential design factors. Although a simple span bridge can experience transverse cracking in its bridge deck, the cracks often only develop on the bottom of the deck where deicing chemicals and water are not likely to penetrate. A continuous span bridge, however, causes tensile forces in the top of the deck and thus can produce transverse cracking at the top of the deck over the interior supports(1). These cracks can be detrimental to the deck. Deck thickness plays a role in the amount of drying shrinkage that occurs and the stresses that develop from shrinkage and thermal changes. A thinner deck has a higher surface area to volume ratio. The higher ratio leads to more drying shrinkage(3). Thinner decks also develop higher stresses from thermal changes. The only disadvantage of a thicker deck is that thermal energy does not conduct from the top to the bottom as fast as a thinner deck, causing a non-uniform stress distribution to occur(9). The curvature in the deck that occurs from the non-uniform stress distribution has less of an effect on cracking than the advantages that the thicker deck brings. If the top and bottom reinforcing bars are aligned vertically, additional stresses will develop due to the weakened cross section in that area. Staggering the alignment of the top and bottom reinforcement will make the deck less susceptible to cracking in these areas(9).

Girder type often has more influence on deck cracking than the deck thickness and influences cracking from thermal changes and drying shrinkage. Of the two main types of girders, a concrete girder usually has less transverse cracking than a steel girder. This can be attributed to the similar coefficient of thermal expansion of the concrete girder compared to the deck and beneficial shrinkage and creep of the concrete girder(11). When the deck is poured, heat of hydration from the deck concrete causes steel girders to expand much more than the concrete girders, due to a higher coefficient of thermal expansion of the steel girders and a much larger heat sink created by the larger mass of the concrete girders. When temperatures cool to normal, the steel girders try to compress the bottom of the deck and cause a convex upward curvature in the deck, creating tensile stresses and transverse cracking on the top of the deck. After the concrete has hardened, seasonal temperature cycles affect steel girders more and diurnal cycles affect concrete girders more because of the larger mass of the concrete girders causes them to expand and contract at a slower rate than the steel girders. Larger steel girders can also react slowly to diurnal temperature changes if they have enough mass; therefore, girder size influences cracking. In addition, one study stated that the relative stiffness of the deck with respect to the girder stiffness is more important than the girder type(10). If a stiffer (larger) girder supports a less stiff (thinner) deck, the girder will restrain the deck more than a smaller girder on a thicker deck, causing more severe transverse cracking(12).

Other minor design factors that influence transverse cracking are: concrete cover, concrete slump, rebar size, bar coating, girder spacing, girder bearing, span length, width of deck, skew, and wearing surface. As discussed in Section 1.1.5, settlement cracking increases with increasing bar size and decreasing concrete cover. To reduce settlement cracking, reduce the maximum bar size to No. 5 and increase concrete cover to at least 1.5 inches(9). The concrete cover should not be increased beyond 3 inches, since the top reinforcement helps transmit the tensile stresses along with the concrete in the top of the deck(4). Epoxy-coated bars can reduce the bond between the concrete and the bars, reducing the efficiency of the top reinforcement in transmitting tensile stresses and increasing the amount of transverse cracking compared to uncoated bars(9). The epoxy-coated bars help keep the rebar from corroding, however, and allow the deck to have more severe cracking before replacement of the deck is required. Spacing of the girders affects how much the deck is restrained. The closer the girders are together the more restraint they provide to the deck(9). The bearing pads provide additional restraint in the girders. According to one study, elastomeric bearing pads restrain the girder rotation more than steel bearing pads, but the sample size of the bridge survey was small and other factors may have influenced the results(10). In addition, longer spans and wider decks can increase transverse cracking, since there is a greater amount of concrete that is available to shrink. Studies show that spans beyond 90 ft or deck widths wider than 70 ft increase the amount of transverse cracking[9]. Another design factor that affects cracking is skew. Skew does not increase the amount of cracking until it is greater than 30° [9]. One researcher indicated that bridge decks with latex wearing surfaces had reduced cracking compared to conventional concrete wearing surfaces, but due to the small sample size in the study any general conclusions should be looked at with caution[10].

1.2.3 Material Properties

The dominant material parameters are cement content, aggregate type and quantity, air content and deck concrete modulus of elasticity[11]. The following subsections discuss these parameters as well as a few more material properties that have an effect on the amount of transverse cracking.

Aggregate

According to one researcher, aggregate was the most important concrete component affecting cracking[9]. Since the drying shrinkage of aggregate is only about $^1/_6$ to ¼ that of the cement paste, increasing the aggregate quantity reduces the amount of shrinkage and cracking in the concrete[3]. The type and size of the aggregate influences how much the shrinkage is reduced. Larger aggregate in a dense gradation occupy a greater volume in the mix that would normally be occupied by cement and water without affecting slump[3,12]. In addition, rough texture and/or flat and elongated particles increase the aggregate absorption, requiring more water to reach the desired slump and more water means more cracking.

Although the absorption of aggregates is measured and the additional water required is controlled by the maximum water-to-cement ratio (w/c) in the specifications, high absorption aggregates tend to shrink appreciably themselves and be more compressible[1]. Less compressible (rigid) aggregates such as dolomite, feldspar, granite, limestone, and quartz restrain shrinkage of the cement paste, creating extremely small strains throughout the matrix and micro cracking (preferred) between the aggregate particles[3]. However, the higher rigidity of the aggregates increases the modulus of elasticity and decreases creep of the concrete, increasing stress build up and partially offsetting the benefits of microcracking[4, 12]. Aggregates with low coefficients of thermal expansion, such as limestone or basalt, can also decrease the amount of cracking[4].

Cement

The type of cement influences the heat of hydration and shrinkage. By using slower strength gain cements such as Type II, the heat of hydration and thus the risk of cracking from thermal stresses are reduced. Both Type I and Type III have high early strength gain and increase the risk of cracking[(12)]. The heat of hydration is also increased when cements with finer particles, higher sulfate content, and higher tricalcium silicate content are used[(9)]. After the initial cure period, the type of cement influences the level of shrinkage stresses in the concrete. Low alkali content cements tend to have lower modulus of elasticity and higher creep, reducing the risk of cracking. Studies on shrinkage-compensating cements, while not conclusive, indicate the potential for promising results[(12)].

Paste

The mixture of water and cement constituents in fresh concrete is considered the paste. Since the majority of shrinkage takes place in the hydrated cement paste, reducing the paste volume will decrease drying shrinkage[(6)]. Reducing the paste volume will also decrease the heat of hydration in the concrete[(4)]. Schmitt and Darwin suggest that the paste content be limited to 27.5 percent[(13)].

Admixtures

Concrete used for bridge decks typically consist of the following admixtures: 1) air entraining agents (AEA), 2) retarders/accelerators, 3) silica fume, 4) fly ash, 5) water reducing agents (WRA), and 6) shrinkage reducing agents (SRA).

Retarders are used with continuous deck casting and are used to slow the early strength gain. They increase susceptibility to plastic shrinkage cracking, but can reduce temperature gain during early hydration reducing thermal stresses. Accelerators are rarely used but occasionally are specified. Although they help speed up the construction process, allowing forms to be removed sooner, they increase the early modulus of elasticity and early temperature rise causing thermal and shrinkage stresses to develop and increasing the probability of cracking[(9)].

Silica fume is a by-product from the production of silicon metal or ferrosilicon alloys in electric arc furnaces. It consists of fine particles having surface area to unit mass ratio 100 times finer than Portland cement. Silica fume mixes with cement paste and aggregate to form a dense material with increased strength and decreased permeability; however increasing the content of silica fume beyond 6 percent has diminishing returns on the reduction of permeability[(12, 13)]. The finer silica fume particles cause the cement to hydrate at an increased rate, increasing the early modulus of elasticity, lowering creep, and increasing early temperature rise[9]. All of these cause higher stresses to develop in the concrete. The tighter pore structure also causes increased autogenous shrinkage due to increased self-desiccation and reduces the rate of bleed water rise, increasing the risk of plastic shrinkage cracks[12]. If a 7-day AASHTO “Water Method” cure is used followed by a membrane cure, the probability of plastic shrinkage cracking can be minimized[13].

Class F and Class N fly ash can be used to reduce the rate of hydration of the cement paste. This decreases early strength and temperature rise and increases creep, reducing stress build up in the deck[9]. Water reducers can decrease the amount of drying shrinkage significantly if they are used with the intent to lower the total water content, decreasing the available absorbed water that can evaporate, and not just increase workability without changing the water-to-cement ratio. A high-range water reducer can reduce the total water content by 100 lb/yd^3. This can reduce drying shrinkage by 30 percent at 84 days without affecting any other proportions or properties of the concrete[3].

As discussed in Section 1.1.2, absorbed water in the concrete separates the C-S-H particles. When the absorbed water evaporates, drying shrinkage occurs. Shrinkage-reducing admixtures reduce drying shrinkage by relaxing the surface tension of the pore-water menisci as the absorbed water evaporates, reducing the capillary tension that develops in the pores of the concrete[3]. Adding 1 percent and 2 percent SRA (by weight of cement) can reduce crack width by 33 percent and 66 percent, respectively, at 10 to 50 days[14].

Water Content, Cement Content and Water-to-Cement Ratio

Higher total water content increases the thickness of the absorbed water layer between the C-S-H particles, increasing the amount of drying shrinkage. The excess water can also be detrimental to concrete strength, durability, and volumetric stability[3]. Although the total shrinkage is increased, the potential for cracking may not increase because higher water content usually increases the water-to-cement ratio, which equates to more creep. Research shows that high cement content and low w/c concretes are at a greater risk to cracking than low cement content and high w/c concretes[9]. However, lower w/c concretes usually also have more autogenous and plastic shrinkage problems[12]. Numerous studies show that high cement content, above 658 lb/yd^3, has more of an influence on cracking than high water content or high water-cement ratio[9, 10, 11, 12, 15, 16].

Air Content

To allow free and absorbed water in the concrete matrix to expand when it freezes, an air entraining agent is used to create air voids that the water can expand into. The volume of the matrix occupied by air is not susceptible to drying shrinkage; therefore increasing the air content decreases the risk of drying shrinkage cracking. A significant decrease in shrinkage is observed in concretes with air content greater than or equal to 6.0 percent[15].

Slump

Increased settlement cracking occurs with increasing slump; however proper consolidation of the concrete can greatly decrease the amount of settlement cracking that occurs[9]. Other types of cracking are not affected by slump, but the higher water content usually associated with higher slump increases the ultimate shrinkage.

Creep

Restraint of the deck is the number one cause of cracking in concrete decks. Creep reduces the tensile stresses that develop from the restraint of shrinkage and thermal effects and thus reduces the amount of transverse cracking[9]. The rate at which strain is applied has a larger impact on the extent of deck cracking than the ultimate strain from shrinkage and thermal effects because, given time, creep can mitigate the stresses that develop from the restrained strains[11].

Concrete Compressive Strength

High concrete compressive strength is typically associated with more transverse cracking. This can be attributed to increased cement contents, paste volume, early modulus of elasticity, hydration temperature and lower creep. Doubling the compressive strength reduces the allowable strain before cracking occurs by half. The reduction in allowable strain is from a 75 percent decrease in creep and 42 percent increase in modulus of elasticity with only a 42 percent increase in tensile strength. This suggests that creep and modulus of elasticity have a greater influence on cracking tendency than tensile strength[9].

Modulus of Elasticity and Poisson's Ratio

The modulus of elasticity is the linear correlation between stress and strain; therefore, the higher the modulus the less strain the concrete can handle before the stresses surpass the rupture strength and cracking occurs. During the first 3 to 5 hours after the fresh concrete begins to harden, the modulus of elasticity increases faster than the concrete strength. At this time, the concrete is more vulnerable to strain/stress increases[16]. The modulus of elasticity of the concrete can be lowered by using aggregate with lower modulus of elasticity and lower compressive strength concrete. Poisson’s ratio has little effect on thermal and shrinkage stresses; however deck stresses generally increase with increasing Poisson’s ratio[9].

Concrete Coefficient of Thermal Expansion

Temperature changes in a deck create stresses that are linearly proportional to the coefficient of thermal expansion. Reducing the coefficient of thermal expansion reduces the strains/stresses applied to the deck from thermal effects, especially from diurnal temperature changes. Since cement paste has a coefficient of thermal expansion (10 to 11 microstrain per °F) 2 to 3 times greater than most aggregates (see Table 1-2), the easiest way to decrease the coefficient of thermal expansion of concrete is to increase the aggregate content[9]. Concrete usually ranges between 4 to 7 microstrain per °F[12].

Table 1-2: Coefficients of Thermal Expansion for Common Aggregate Types[9]

	Granite	**Basalt**	**Limestone**	**Dolomite**	**Sandstone**	**Quartzite**
α **(με/F)**	4-5	3.3-4.4	3.3	4-5.5	6.1-6.7	6.1-7.2

1.2.4 Construction Techniques

Construction techniques can have a significant effect on plastic shrinkage, drying shrinkage, plastic flexure, and settlement cracking. To reduce plastic shrinkage, the evaporation rate at the time of pour should be at a minimum, below 0.2 $lb/ft^2/hr$ for normal weight concrete or 0.10 $lb/ft^2/hr$ for concrete with water to cement (w/c) ratios of 0.40 or less. This can be achieved by waiting for ideal conditions, putting up windbreaks, shading the deck, or applying a fog mist or evaporation retarder film as soon as possible after pouring[9]. Delayed finishing of the deck can also increase the amount of plastic shrinkage[12].

Like plastic shrinkage, drying shrinkage is affected by the evaporation rate, but long-term evaporation is a greater factor. If corrugated stay-in-place (SIP) steel forms are used, the long-term evaporation is less on the bottom of the deck, causing a linear shrinkage gradient[9]. As with diurnal temperature changes, a linear gradient causes tensile stresses and thus transverse cracking in the top of the deck. If proper curing procedures are not used, evaporation can increase the rate of drying shrinkage on the deck. Although the ultimate drying shrinkage is the same, the rate of shrinkage has a greater effect on the amount of transverse cracking because creep does not have time to dissipate the stresses that develop. To reduce the rate of drying shrinkage, 14 days of moist curing using wet burlap after applying a fog spray should be used[9].

Drying shrinkage cracking can also be experienced on decks in staged construction because the first stage of the deck that was poured already finished the majority of its shrinkage before the second stage was poured. When the freshly poured deck tries to shrink, the older portion of the deck restrains the shrinkage causing transverse cracks to form in the new deck(9).

There are numerous reasons that flexural cracking can occur in a bridge deck. While the concrete is plastic, both the sequence of the pour and the deflection in the formwork from the dead load can cause cracking. If the negative moment region has been poured, followed by the positive moment region, the deflection in the formwork will cause excessive curvature in the negative moment region. Since the negative moment region was poured first, the concrete may have already started to harden, but the modulus of rupture is extremely low and the curvature causes tensile stresses and transverse cracking to form in the negative moment region(9). After the concrete has hardened and is still young, it has not reached design strength and flexural cracking can occur if the formwork is removed prematurely(16). Cracking can also occur if heavy construction loads are applied to the deck at early ages; however the cracking that occurs from flexural loading of the deck is minimal compared to other types of transverse cracking on the bridge(9).

Settlement cracking is one of the easiest types of cracking to avoid if proper construction techniques are used. By properly vibrating the concrete deck, the plastic concrete will already be settled by the time it starts to harden and thus will not form any planes of weakness because of subsidence of partially hardened concrete. Providing a minimum of three vibrators for placement rates of 30 yd^3/hr and revibrating after initial consolidation with a vibrating screed can both help reduce settlement and plastic shrinkage cracks(9).

1.3 Actions to Mitigate Transverse Cracking

Based on the literature review, the following design and construction parameters are suggested. Some of these suggestions may significantly affect other aspects of bridge performance, so the engineer should evaluate the effects of the modification before implementing them. If an evaluation of the influence of the transverse cracking mitigation modifications is needed on a project, then the quality control forms should include wind velocity, humidity, curing method, curing period, and placement length. In addition, during the first year after cracking, all cracks should be mapped (location, length, width) and then sealed(12).

1.3.1 Bridge Design

Based upon the information gathered in the literature review, the design recommendations presented in Table 1-3 may reduce the risk of transverse cracking in the bridge deck. Please note that these recommendations were gathered from other states' documents and may or may not be applicable to specific Idaho and ITD projects.

Table 1-3: Bridge Design Recommendations.

Design Detail	Recommendations
Concrete Cover	• Specify concrete cover between 1.5 in. and 3.0 in. (preferably minimum cover of at least 2 inches)[1,9]
Deck Reinforcement	• Specify top reinforcing bars (longitudinal and transverse) #5 or smaller and spaced less than 6 inches on-center[16] • Avoid alignment of the top and bottom reinforcement[12] • Place top longitudinal steel above transverse steel[9,12] Note: This recommendation would prohibit the use of the AASHTO Empirical Method. AASHTO LRFD Section 9.7.2.5 requires that the outermost layers be placed in the direction of the effective length.[19]
Deck Thickness	• Specify a deck thickness greater than 8.5 inches[9]
Concrete Strength	• Avoid much higher concrete compressive strengths than design by possibly specifying a maximum concrete strength[10] • Avoid fast strength gain in the deck by specifying reasonable 7-day and 28-day strengths and allowing 56 days to arrive at the design strength[12]
Girders	• Avoid tension in prestressed concrete girders[12] • Reduce restraint of the deck by increasing girder spacing, using deck expansion joints (or using simply supported spans) and avoiding restraint of girder end connections (such as integral girders and abutment)[4] • Use shear connector configurations with fewer number of rows, smaller diameter studs, and shorter length studs[4]
Deflection	• Avoid satisfying the deflection requirements by a large margin, such as designing extremely rigid girders, to allow for more compatible deck/girder stiffness[10]
Skew	• Limit skew of the girders relative to the abutments to less than 30°[9]

Note that these recommendations only address transverse cracking, not the overall bridge performance. For example, the recommendation to use simply supported spans instead of continuous spans would result in larger deflections or would require deeper sections. As another example, the recommendation to limit deck restraint could require the use of expansion joints, which are maintenance intensive.

1.3.2 Concrete Materials

The proportions and type of materials used in the concrete mix can have a significant effect on the hydration temperature, overall drying shrinkage, and speed and magnitude of strength gain, thus affecting transverse cracking. Table 1-4 lists recommendations to help reduce cracking associated with concrete materials. These recommendations are listed in order of decreasing influence. Please note that these recommendations were gathered from other states' documents and may or may not be applicable to specific Idaho and ITD projects.

Table 1-4: Concrete Mix Design Recommendations.

Variable	Recommendations
Aggregate	• Maximize coarse aggregate content (1800 to 1850 lb/yd^3). Use larger size (up to 1.5 in.); densely graded, rigid, low-shrinkage, low coefficient of thermal expansion, and high conductivity aggregates [3,4,5]
Cement	• Avoid finely ground cement[1] • Use Type II Cement[1] • Limit the cement content to 470 lb/yd^3[12]
Water Content	• The water content should be kept below 300 lb/yd^3[3]
Water/Cement Ratio	• The water/cement (w/c) ratio should range between 0.40 and 0.45 (preferably a ratio of 0.40)[12] • The water and cement content should not exceed 27 percent of the total volume of concrete[16]
Slump	• Specify a slump around 2 inches[16]. If a water-reducing admixture is used to reduce the water content, slumps up to 8 in. can be specified
Shrinkage Reducing Admixtures	• Add between 1.0 percent and 2.0 percent Shrinkage Reducing Admixture (SRA) by weight of cement[14]
Air Entrainment	• Specify air content of 6 percent or higher by volume[16]
Fly Ash	• Replace up to 28 percent (20 percent recommended) of cement by weight to control strength growth[9]
Silica Fume	• Limit to 6 percent by weight of cement [12] Note: Will increase heat of hydration and promote early age cracking if not moist cured immediately after concrete placement.

Many of these recommendations have been incorporated into ITD's bridge design and construction specifications.

1.3.3 Bridge Construction

Construction procedures greatly influence the amount of deck cracking. The procedures in Table 1-5 are some suggestions on procedures that could reduce cracking. Many of these recommendations are contained in ITD bridge design and construction specifications. Please note that these recommendations were gathered from other states' documents and may or may not be applicable to specific Idaho and ITD projects.

The following rules of thumb for deck placement sequence should also be followed *(16)*:

1. Whenever feasible, the entire deck should be poured in one placement,

2. If the bridge is composed of simple spans (and multiple placements are needed) then pour each span in 1 placement,

3. If rule 2 applies but more than one placement is needed, divide the deck longitudinally and pour in 2 equal placements,

4. If rules 2 or 3 apply but the entire span cannot be poured in one placement, pour the center portion as large as possible first.

5. If the bridge is continuous (and multiple placements are needed) then pour the center of positive moments first followed by the interior supports 72 hours later.

Note that the ITD construction specifications often contain requirements that are more restrictive than the recommendations listed in Table 1-5. For example, due to the low humidity which often occurs at construction sites in Idaho, the ITD construction specifications requires a longer curing time than that recommended by AASHTO.

Table 1-5: Construction Procedure Recommendations.

Construction Procedure	Recommendations
Placement	• Casting Sequence: Pour concrete deck at one time, within the limitation of maximum placement length based upon drying shrinkage • If multiple placements are are required, place concrete in positive moment regions first, and observe 72 hour delay between placements • Use of stay-in-place (SIP) forms causes linear shrinkage gradient, which produces tensile stresses on upper deck surface[9] • Form removal after specified strength has been achieved • Place the deck when temperatures are between 45° F and 80° F (If casting must occur during temperatures outside this range, see the procedures for cold or warm weather placement)[12] • Pour when the daily temperature fluctuation is less than 50° F[4] • Maintain the girder/deck differential temperature under 22°F for at least 24 hours after the concrete is placed[1] • Avoid placement when the evaporation rate is 0.20 lb/ft^2/hr for normal concrete and 0.10 lb/ ft^2/hr for concrete with w/c of 0.40 or lower[12] • Avoid casting during high winds and use windbreaks when applicable • Use a minimum of 3 vibrators for placement rates of 30 yd^3/hr or more • Revibrate with a vibrating screed • Apply mist water or an evaporation retarder film immediately after screeding[9] • Dampen the forms and or supporting structure as well as reinforcing steel with water
Warm Weather (above 80°F) Placement	• Place the concrete during the evening • Use cold mixing water, possibly by incorporating ice • Keep aggregate cool by shading them • Shade the deck from solar radiation if possible[1]
Cold Weather (Below 40°F) Placement	• When possible, pour the deck during periods of sunny weather[16] • Maintain temperatures between 55° F and 75° F under insulated concrete covers to reduce drying and heat of hydration temperatures[12] • If the deck is insulated, heat the air underneath the deck[1] • Use warm mixing water • After curing is complete, gradually lower the concrete temperature to the ambient air temperature[1]
Finishing	• Complete surface finishing and texturing as soon as possible to allow the final cure of the deck • Except at the edge of the deck, do not allow hand finishing unless it is approved by the engineer[12] • Perform grooving using a diamond saw instead of rake tining[9]
Curing	• Apply white-pigmented curing compound uniformly in 2 directions when the bleed water diminishes but before the surface dries[9] • Protect concrete with a protective barrier, such as wet burlap, curing membranes, vinyl covers, etc.[16] • Use The AASHTO "Water Method" for a minimum of 7 days[13]. (Note: ITD uses 10 days)

RATIONALE BEHIND TESTS AND MEASUREMENTS

To help determine the causes of concrete bridge deck cracking, information from the US 95 bridge over the South Fork of the Palouse River(17) was compared to recommendations of the literature review. The US 95 bridge over the South Fork of the Palouse River was the first in the state to specify the use high performance concrete (HPC) for the bridge deck. For comparison purposes, the data for environmental conditions, construction techniques, material properties, deck strains, and deck temperatures were obtained.
Section 2.1 and 2.2 discuss the rationale behind the material tests performed and the locations of the strain and temperature gauges, respectively.

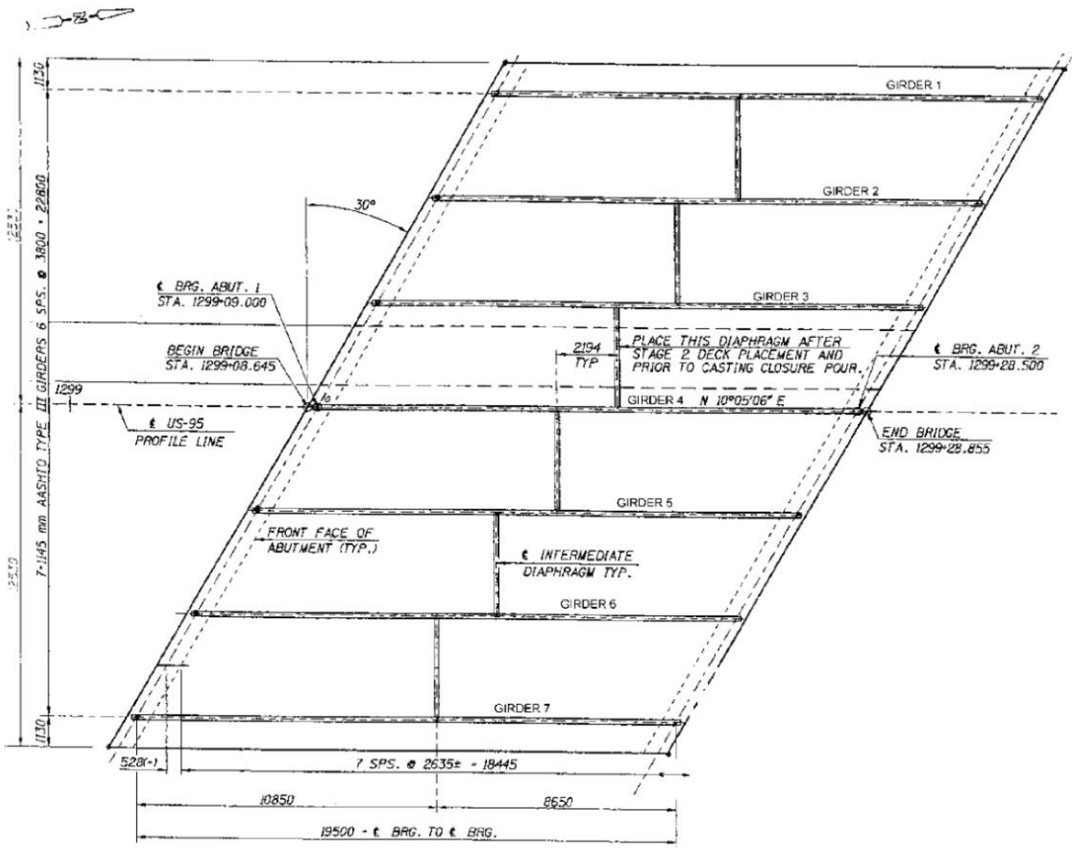

Illustration 2-1: U S 95 Bridge Plan.

2.1 Materials Tests

To help determine the cause of transverse cracking in the bridge deck, the material properties of the deck concrete needed to be determined. A list of the standard tests that were performed by researchers at the UI on the concrete is shown in Table 2-1.

Table 2-1: **Standard Tests Performed**

Standards Organization	Test Designation	Test Title
AASHTO	T22	Compressive Strength of Cylindrical Concrete Specimens
AASHTO	T160	Length Change of Hardened Hydraulic Cement Mortar and Concrete
AASHTO	T277	Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration
ASTM	C469	Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression
ASTM	C496	Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens.
ASTM	C512	Standard Test Method for Creep of Concrete in Compression
ASTM	C666	Resistance of Concrete to Rapid Freezing and Thawing (Method A)
ASTM	C672	Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals
ASTM	C944	Standard Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating-Cutter Method
CRD US ARMY Corps of Engineers	C39	Test Method for Coefficient of Linear Thermal Expansion of Concrete
NCHRP	380	Proposed Standard Method for Testing Cracking Tendency of Concrete

As will be discussed in later sections, the bridge deck was outfitted with strain gauges and thermocouples at various points. The material properties obtained from the tests shown in Table 2-1 provide a correlation between the recorded strain and the stress in the deck. Using AASHTO T160 and CRD C39 (with the thermocouple data), the recorded strain data was broken down into the component caused by shrinkage and thermal effects. Since strain from creep does not cause any stress, ASTM C512 data was used to subtract out the portion of strain caused by creep. With the adjusted strain value, the stress at gauge locations was calculated using the modulus of elasticity of the deck concrete, determined with ASTM C469. The stresses in the deck were then compared to the ASTM C496 data, to identify potential locations of cracking based on the tensile capacity of the concrete. The expected time of cracking is established using NCHRP 380 data(4). The compressive strength of the concrete, determined by AASHTO T22, helps determine normal values of modulus of elasticity and tensile strength at various times so that a comparison can be made to the actual test results. The correlation between compressive strength and modulus of elasticity and the tensile strength and compressive strength are shown below.

Equation 2-1: Modulus of Elasticity of Concrete

$$E = 57000 * \sqrt{f`_c},$$

Equation 2-2: Tensile Strength of Concrete

$$f_{ct} = 6to7 * \sqrt{f`_c},$$

where E is modulus of elasticity (psi), $f`_c$ is concrete compressive strength (psi), and f_{ct} is concrete tensile strength (psi). In addition to providing correlations to other tests, the T22 test shows the rate of strength gain for comparison between mixes. The faster the strength gains, the higher the risk for transverse cracking in the deck. The T22 test is also performed at closer intervals than C469 and C496 tests, so a correlation between the two tests can be made if values for modulus of elasticity or tensile strength are needed for times other than when they were tested.

The remaining tests, which include: AASHTO T277; ASTM C666, ASTM C672, and ASTM C944, provide information on the durability of the deck concrete. Although there is no direct correlation between these test results and transverse cracking, they establish how the concrete will react to deicing chemicals (C672), if chloride ions will penetrate to the reinforcement (T277), how the concrete will handle freeze thaw cycles (C666), and how well the concrete will handle abrasion from traffic loads (C944). If the bridge deck begins to deteriorate over time, these tests may be used to establish the possible cause. Results of these tests will be presented; however, the only analysis that can be performed is a comparison to the ITD specifications, where applicable, or a statement on whether the concrete is above or below average for that test.(17) *I*t is important to note that the ASTM C672 test was performed using a concentration of 28.5 percent, by weight, of magnesium chloride (standard concentration used on Idaho roads) instead of 4 percent, by weight, calcium chloride as specified in the standard testing procedures for this test.

Magnesium Chloride was used because it is the preferred deicing chemical for the state of Idaho and would be what the deck is exposed to in actual conditions. All other tests were performed as outlined in the AASHTO or ASTM standard testing manuals.

2.2 Deck Instrumentation

Due to the high cost of vibrating wire (VW) strain gauges, limitations were set on the number of gauges available for mounting on the bridge deck. Section 2.2.1 and 2.2.2 discuss the rationale behind mounting the gauges and selection of gauge mounting locations, respectively.

2.2.1 Gauge Mounting

Vibrating wire strain gauges were chosen over electrical resistance strain gauges because the resistance in the lead wires does not affect readings and gauges can be detached from their lead wires without resetting the strain reading. In addition, the wire to gauge interface is also a thermocouple, so temperature data is recorded in the same locations as strains. Since the strain varies throughout the deck, an embedment gauge was attached to the top reinforcement and a surface mount gauge was attached to the bottom of the deck directly below the embedment gauge. For analysis, a linear distribution of strain and temperatures between the top gauge and bottom gauge was assumed. A gauge was also attached to the girder near the surface mount gauge to measure the difference in strain between the deck and the girders, determining the restraint the girder applies. Illustration 2-12 shows the configuration of gauges near a girder. Gauge configurations between girders are similar but there is no girder mount gauge.

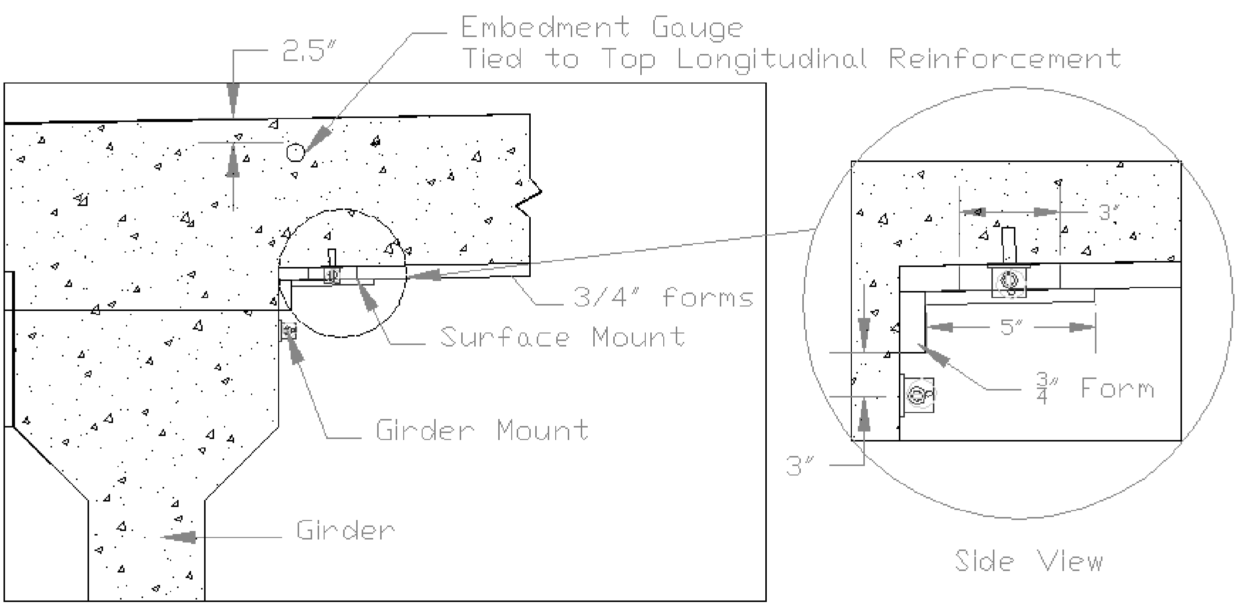

Illustration 2-2: Gauge Mounting Configuration.

Shown in Illustration 2-3 is the attachment of the embedment gauge to the top reinforcement as well as the top of the surface mount attachment. The embedment gauge is attached to the epoxy-coated top longitudinal reinforcement using wire ties. The picture also shows the portion of the surface mount gauge that will be attached to the concrete deck. Using 1½ x ⅜ in. bolts attached to the bottom of the gauge, the surface mount gauge can be in place before the deck is poured so that strains and temperatures can be measured while the concrete is in a plastic state. Illustration 2-4 shows attachment of the girder mount and surface mount gauges. The girder mount gauges are attached to the girder prior to the deck pour using high strength epoxy. For the surface mount gauges, a 3 by 8 in. hole was cut out of the ¾ in. plywood forms at the desired location. A wood assembly was used to hold the gauge in position while the concrete was being poured. The assembly was designed to screw into the bottom of the formwork and allow movement of the gauge in the longitudinal direction. After the concrete hardened, the assembly could be removed to allow construction workers to easily remove the formwork without disturbing the gauge.

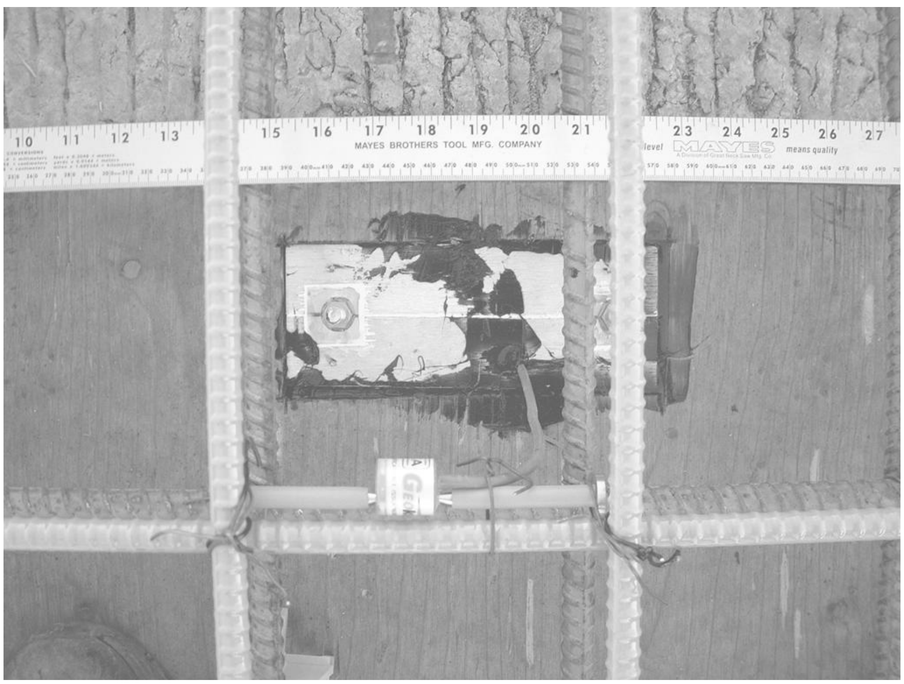

Illustration 2-3: Embedment Gauge Attachment.

Illustration 2-4: Surface Mount and Girder Gauge Attachment.

2.2.2 Location of Gauges

The gauges available for measurement included: 4 girder mount, 6 bottom surface mount, and 6 embedment gauges on both the Stage 1 deck and the Stage 2 deck. An accurate estimate of the strains and temperatures on the deck at any point can be calculated using data readings at the locations shown in Illustration 2-5. The gauges were located where the maximum and minimum strains and temperatures were expected for the Stage 1 deck, which used conventional concrete instead HPC as originally intended. The Stage 2 deck gauges were added when HPC was used only for the Stage 2 deck. Note that the Stage 1 deck gauges are located in closest to the acute corner of the deck, while the Stage 2 deck gauges are located closest to the obtuse corner of the deck. The other strains and temperatures in the deck are estimated based on measurements at these locations. Temperatures in the deck are expected to reach a maximum near the abutments, due to the large volume of concrete at his location causing high heat of hydration temperatures. Minimum temperatures are expected to occur near center span between the girders followed by areas near center span close to a girder. Gauges near center span are expected to have the highest strains, while gauges between girders allow better isolation of the strains caused by girder restraint. The gauges near the exterior girders are expected to have lower temperatures due to the greater dissipation of heat near the edges of the deck.

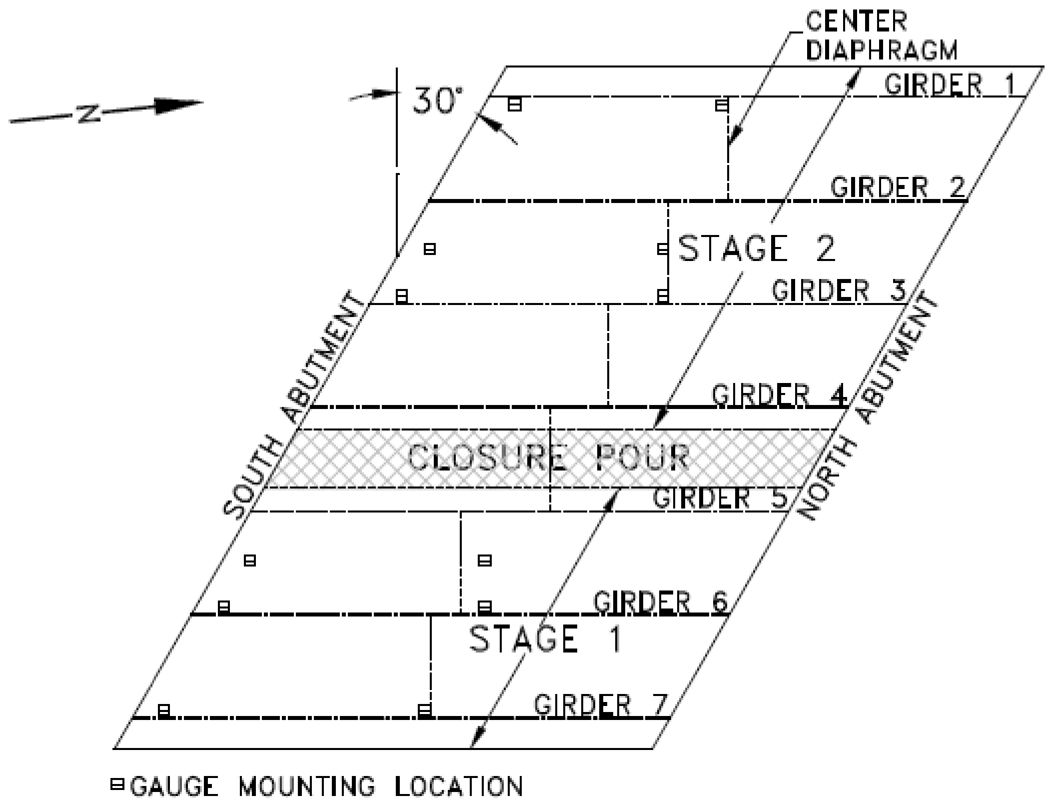

Illustration 2-5: Approximate Location of Vibrating Wire Strain Gauges.

3. CONCRETE MATERIALS ANALYSIS

Prior to pouring of the bridge deck for Stage 1, the contractor performed tests on multiple test mixes for high performance concrete (HPC). For Test Mixes 1 and 3, the University of Idaho (UI) performed its own material testing together with the contractor's testing. These tests allowed the UI to verify its testing procedures and equipment. Since a HPC mix design was not approved before the first stage of the bridge deck was poured, as discussed in Section 3.1, the data obtained from the test mixes allowed for preliminary comparison of HPC with "conventional" concrete. Section 3.2 discusses the results of the material testing on the deck concrete used in Stage 1 of bridge construction. Section 3.3 summarizes the results of the materials tests presented in Section 3.1 and 3.2.

3.1 HPC Test Mixes

Each test mix design was based on the project specifications[17] shown in Table 3-1. In addition, the specifications state: "5 percent by weight of the total cementitious materials content shall be silica fume. 20 percent by weight of the total cementitious materials content shall be fly ash[17]." Although the design strength of the concrete is 4000 psi, Section 502.03 of the project specifications requires the compressive strength, as tested using AASHTO T22 procedures, to be 1200 psi above the design strength. For the HPC mix, the minimum compressive strength for test purposes must be above 5200 psi to pass. Sections 3.1.1 and 3.1.2 present the test results for Test Mixes 1 through the approved test mix (which was only used on the second stage of the bridge deck), and the mix used on the first stage of the bridge, and compare them to the specifications for this project.

Table 3-1: ITD Specifications for HPC Mix Design[17]

Concrete Class in 100 psi (28-Day)	Maximum Cementitious Materials Content (lb/yd3)	Water to Cementitious Materials Ratio (range)	Slump (in.)	Air Content %	Max Ratio of Fine Agg. to Total Agg. Content	Permeability (Coulombs)	Shrinkage Potential (micro-strains)
40-HPC	583	0.38 to 0.40	1.5-3.5	6.5 +-1.5	0.38	<1500	<400

3.1.1 Specification Comparison

The intention of this project was to be the first bridge deck in Idaho to use high performance concrete (HPC) to reduce bridge deck cracking. The contractor, however, had difficulty producing a mix that met the specifications for class 40-HPC concrete. Table 3-2 presents the results for the first 4 test mixes and the mix design that was finally approved by ITD for use on the Stage 2 deck. The Stage 1 deck used a conventional mix, shown in section 3.2.1, while the Stage 2 deck used what was intended to be the high performance mix. All the mixes tested met shrinkage potential, air content, and slump; however, permeability and compressive strength requirements were rarely if ever passed.

By the time the final test mix was performed, ITD had dropped the permeability requirements in the interest of moving the project along, due to the difficulty the contractor was having in meeting the requirement. Test Mix 3 was the closest to meeting the entire mix design requirements. The compressive strength easily passed the 5200 psi requirement and the water-tocement ratio and cement content came in at the upper end of the limits. Fine aggregate to total aggregate ratio was a bit high, but the permeability requirement was only about 1100 coulombs above the specifications compared to the next best mix, which was more than 2400 coulombs above. An outside concrete plant that did not win the bid for concrete supplier made the concrete according to specifications (batched the mix) for Test Mix 3, at the request of the contractor, and therefore it could not be used on the project. The decreased permeability and increased compressive strength were attributed to the higher-grade aggregate used in Test Mix 3. The aggregate used in Test Mix 3 was a round river rock from the Lewiston Valley as opposed to the highly porus, crushed basaltic aggregate that was used on Test Mix 1 and 2. The test mix designs batched after Test Mix 3 used the Lewiston Valley aggregate source. Unfortunately, the approved concrete supplier could not reproduce the test results of Test Mix 3 in subsequent test mix designs, such as Test Mix 4. The final approved mix design was batched about a month before Stage 2 of the bridge deck was poured. Shrinkage potential had not been a problem on other mix designs so it was not tested and the permeability requirement had been dropped. All of the test requirements were met except the compressive strength. Although it was 120 psi under the requirement, ITD decided this was not a large enough margin to justify rejecting the mix.

Table 3-2: HPC Test Mix Material Test Results

Mix Design	Compressive Strength (psi) (28-Day)	Cement Content (lb/cy)	w/c	Slump (in.)	Air Content (%)	Fine Aggregate /Total Aggregate	Permeability (Coulombs)	Shrinkage Potential (microstrain)
Test Mix 1	4690	619	0.42	3	6.58	0.41	NA	100
Test Mix 2	3500	NA	NA	3.5	6.80	NA	3963	43
Test Mix 3	6440	588	0.40	4	5.60	0.40	2615	70
Test Mix 4	4590	647	0.38	2.5	8.20	0.35	5213	58
Approved (used on Stage 2 deck only)	5080	583	0.40	2.25	5.80	0.37	NA	400

3.1.2 Other Tests Results

For Test Mix 1, UI performed every test listed in Table 2-1 except ASTM C666 and AASHTO T277. These two tests had to be performed by an outside agency and the data they would provide was deemed not important enough to justify having them executed for a test mix. The compressive strength and shrinkage potential are shown in Table 3-2. With the exception of creep and cracking potential, the remaining test results are shown in Table 3-3. Although Test Mix 1 was not used because of failure to meet specifications, the tensile strength and coefficient of thermal expansion are within normal ranges as defined by Wang and Salmon[18].

Results for ASTM Tests C672 and C944 are used mainly for comparison to other test mixes and do not have standard values established; however, a lower scaling rating and grams lost per 2 minute abrasion period indicates a more durable concrete. Test Mix 3 was the only other test mix for which ASTM Test C944 was performed and it had a much lower loss of concrete, compared to Test Mix 1, at 3.3 grams per 2 minute abrasion period, indicating a more durable mix design.

Table 3-3: Additional Test Results for Test Mix 1

Test Designation	Test Results
ASTM C496	7-day; 263 psi: 28-day; 435 psi
ASTM C672	Scaling rating: 1 (very slight scaling with no coarse aggregate showing)
ASTM C944	7.3 grams lost per 2 minute abrasion time
USACE CRD C39	5.52 microstrain per °F

The ASTM C512 (creep) results were obtained from loading three 4 x 8 in. cylinders to 20,000 lb (1600 psi) for the duration of the test. Specimens were loaded at an age of 20 days and were accompanied by three 4 x 8 in. control cylinders used to correct the readings on the creep specimens for thermal changes and shrinkage of the concrete. Corrected creep results are shown in Figure 3-1. In this graph and throughout the rest of the paper, tensile strains are positive and compressive strains are negative. At an age of 275 days the total creep, less the strain from the initial loading, had reached approximately -1300 microstrain.

The NCHRP 380 cracking potential test also provides valuable data on the risk of cracking in the deck concrete. This test was performed for Test Mixes 1 and 3; the results of which are shown in Figure 3-2 and 3-3, respectively. The time of cracking in the test specimen is determined by locating a sudden drop in strain or the rate of strain increase as pointed out in Figure 3-2 and 3-3. Although Test Mix 3 was closer to the specifications and had better test results than Test Mix 1 for most tests, cracking did not occur in Test Mix 1 specimens until around 22 to 24 days, as opposed to 18 days for Test Mix 3. This suggests that if Test Mix 3 was completely restrained it would crack earlier than Test Mix 1.

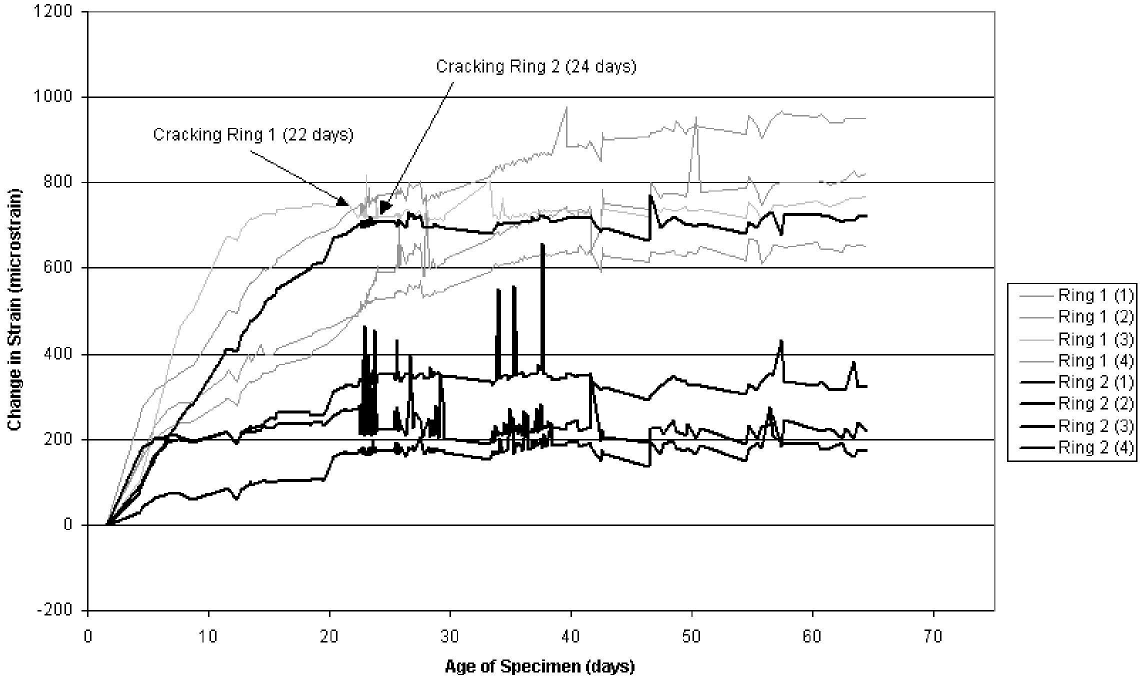

Figure 3-1: Cracking Potential Results for Test Mix 1.

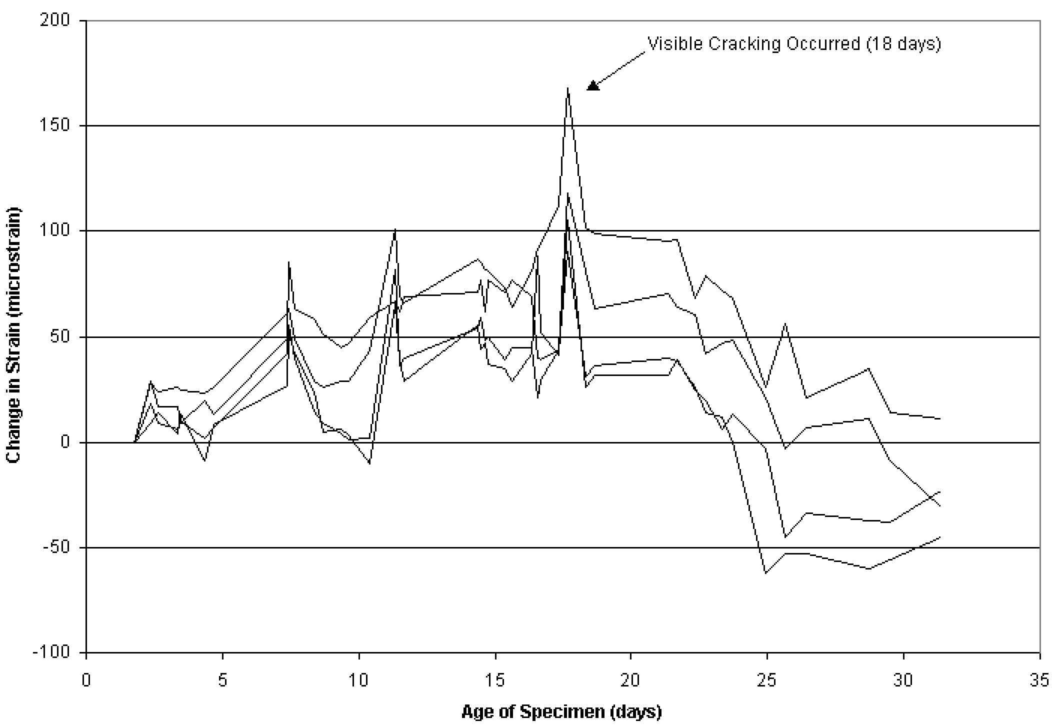

Figure 3-2: Cracking Potential Results for Test Mix 3.

3.2 Bridge Deck Concrete

The tests that were performed for Stage 1 of the deck are listed in Table 2-1. The results of these tests are presented in Sections 3.2.2 through 3.2.4, and a comparison of the mix design to the ITD specifications is presented in Section 3.2.1.

3.2.1 Mix Design

Before Stage 1 of the deck was placed, the contractor attempted to produce a high performance concrete mix design without success. To keep the project moving along, ITD decided to go ahead with the deck pour using an already proven “conventional” concrete mix design. This meant that only the compressive strength, air content, water-to-cement ratio, and slump requirements were used and no silica fume or fly ash would be added to the mix. Table 3-4 presents the average properties of the concrete mix used on the Stage 1 deck.

Table 3-4: Mix Design Properties (Stage 1 Deck)

Coarse Aggregate Content (lb/yd^3)	Cement Content (lb/yd^3)	w/c	Slump (in.)	Air Content (%)	Fine Aggregate/ Total Aggregate	Unit Weight (lb/ft^3)
1868	693	0.39	1.8	5.32	0.37	149

Most of the requirements shown in Table 3-1 are met by this mix design. The water-to-cement ratio, slump, air content, and fine aggregate to total aggregate ratio are all within the limits. Cement content however is 110 lb/yd^3 above the maximum cementitious materials as specified by the project specifications. Comparing these values to the suggestions given in Section 1.3.3, the aggregate content, water-to-cement ratio, water content (270 lb/yd^3), slump, and cement type all fall within the recommendations. Air content is close to the suggested limit of 6 percent, so the increased risk for cracking because of air content should be minimal. The cement content, fly ash, and silica fume values, however, are well beyond the limits for optimal protection against cracking in the concrete. A cement content over 220 lb/yd^3 above the recommended cement content, given in Section 1.3.3, increases the heat of hydration temperatures, response to thermal changes, and drying shrinkage for this mix. In addition, the complete lack of fly ash and silica fume will increase permeability, early modulus of elasticity, early compressive strength, and heat of hydration temperatures.

3.2.2 Compressive Strength, Tensile Strength, Modulus of Elasticity, and Poisson's Ratio

The rate of compressive strength gain is presented in Figure 3-3. As shown, the compressive strength for the Stage 1 deck had nearly attained the specified design strength in 3-days and had surpassed 8000 psi at about 80-days. Although the rapid strength gain is beneficial for construction, it indicates that this mix will have high heat of hydration and high early modulus of elasticity, increasing the risk of transverse cracking. In contrast, the concrete for the Stage 2 deck had a slower gain in strength.

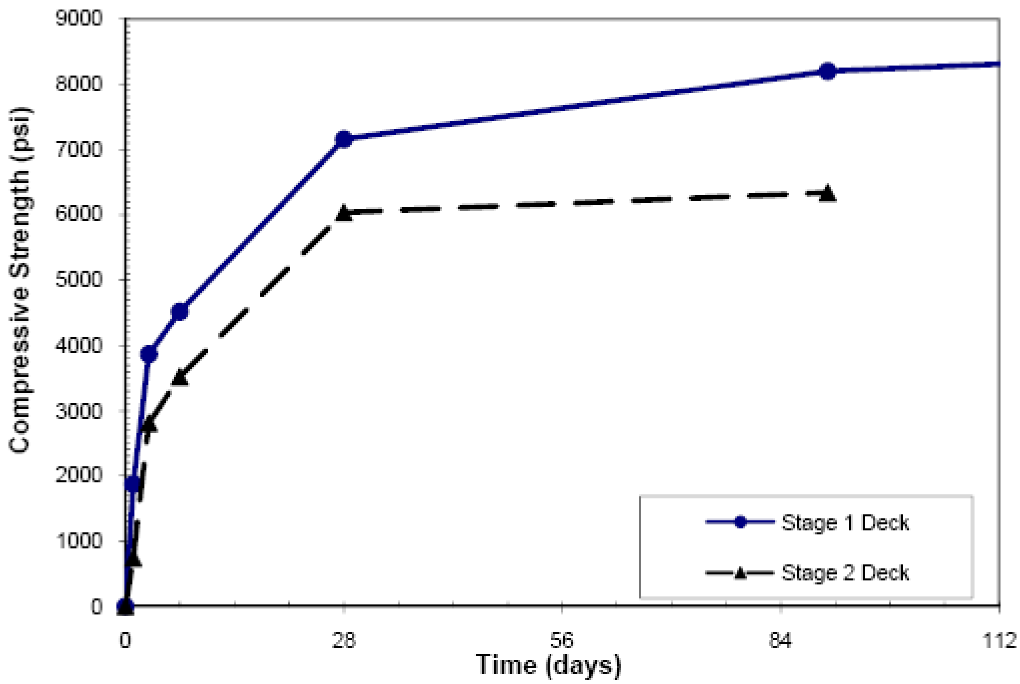

Figure 3-3: Compressive Strength of Bridge Deck Concrete.

The results of the tensile strength, modulus of elasticity, and Poisson's ratio tests are presented in Table 3-5 along with the expected values using the compressive strength and the correlations presented in Section 2.1.

Table 3-5: Tensile Strength, Modulus of Elasticity, and Poisson's Ratio for Stage 1 Deck Concrete

Material Property	7-Day Results		28-Day Results		90-Day Results	
	Actual	Expected	Actual	Expected	Actual	Expected
Modulus of Elasticity (psi)	3940088	3831000	4347978	4822000	5206135	5163000
Poisson's Ratio	0.184	0.11 to 0.21	0.190	0.11 to 0.21	0.198	0.11 to 0.21
Tensile Strength (psi)	517	403 to 470	606	507 to 592	Not Tested	543 to 634

From Table 3-5 it can be noted that the modulus of elasticity is slightly above the expected value for the 7-day and 90-day results and slightly below normal for the 28-day results. Using these values, a second-order polynomial equation was determined, relating the modulus of elasticity to the age of the concrete. This equation is valid only for the deck concrete in Stage 1 and is shown below:

Equation 3-1: Modulus of Elasticity as a Function of Time

$$E = -67.25(A)^2 + 21777.17(A) + 3790943.55\,,$$

where A and E are the age of the concrete (days) and the modulus of elasticity (psi), respectively. Poisson's ratio values are also within the expected range and increase with age as expected. The ratio is on the upper end of the expected range, however, indicating a slightly higher risk of cracking according to the literature review. As shown in Figure 3-5, the modulus of elasticity for the Stage 2 deck concrete was slightly lower than that of the Stage 1 deck concrete. This should somewhat reduce the potential for shrinkage cracks compared to the Stage 1 deck concrete.

The average correlation between the compressive strength and tensile strength based on the values in Table 3-5 was also determined. The tensile strength of the Stage 1 deck concrete at a given age is related to the compressive strength by

Equation 3-2: Tensile Strength of Stage 1 Deck Concrete

$$f_{ct} = 7.4\sqrt{f'_c}\,,$$

where f_{ct} and f'_c are the tensile capacity (psi) and compressive capacity (psi) of the concrete, respectively. The correlation coefficient 7.4 is slightly above the 6 to 7 correlation range stated in Section 2.1, indicating that this concrete has above average tensile capacity. Tensile strength for the Stage 1 deck concrete was also almost twice the tensile strength of Test Mix 1 at 7-days and almost 200 psi greater at 28-days, indicating that the Stage 1 deck concrete could sustain higher stresses before cracking. In contrast, as shown in Figure 4-4, the Stage 2 deck concrete had a significantly lower tensile strength. The Stage 2 deck concrete, while nominally a HPC mix design, at 28-days had a tensile strength that was approximately 100 psi lower than that of the Stage 1 deck concrete. This reduced tensile strength will completely offset any potential reduced tendency to crack due to the lower modulus of elasticity. The tensile strength of the Stage 2 deck concrete at a given age is related to the compressive strength by:

Equation 3-3: Tensile Strength of Stage 2 Deck Concrete

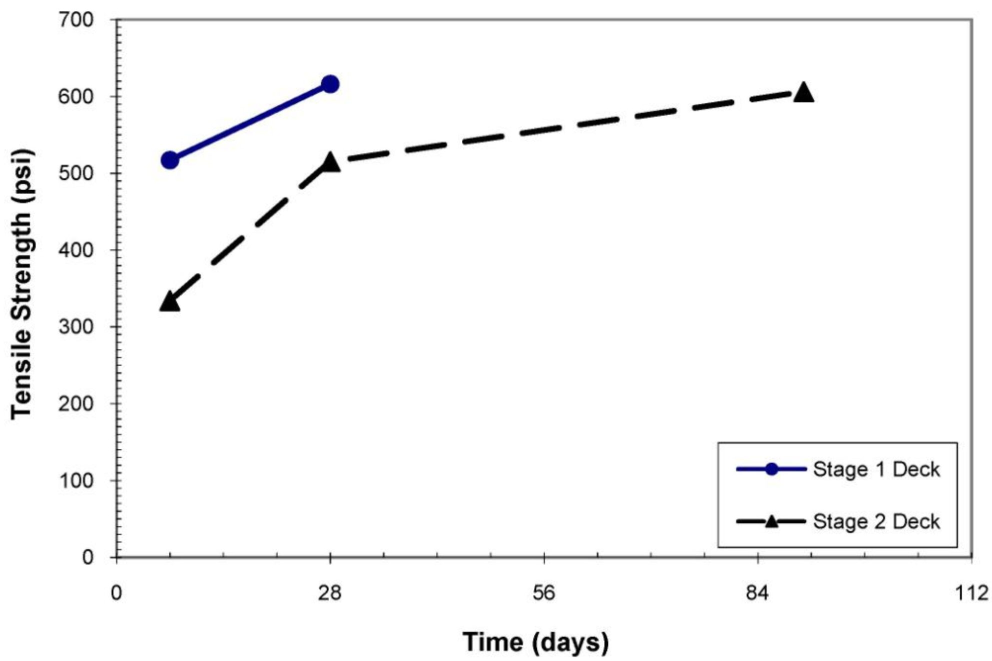

Figure 3-4: Splitting Tensile Strength of Bridge Deck Concrete (ASTM C496).

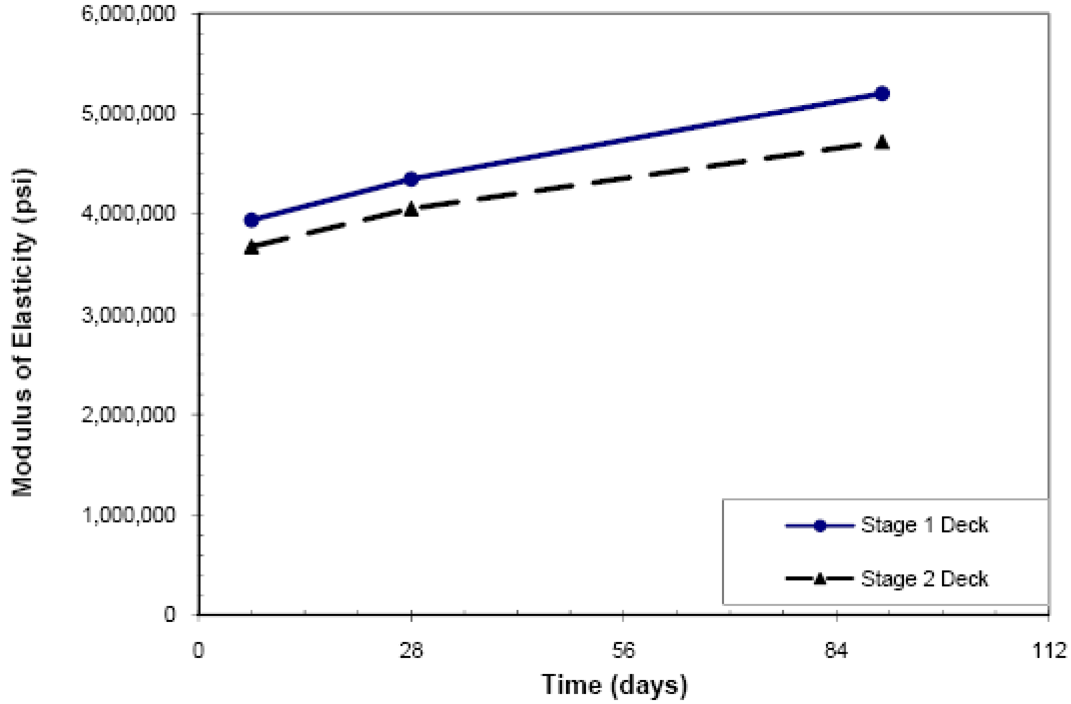

Figure 3-2~~5~~: Modulus of Elasticity of Bridge Deck Concrete (ASTM C469).

3.2.3 Cracking Potential and Coefficient of Thermal Expansion

The coefficient of thermal expansion for the Stage 1 concrete was measured to be 4.37 microstrain/º F. Compared to the range specified in the literature review and the value for Test Mix 1 shown in Table 3-3, the coefficient of thermal expansion of the deck concrete is on the low end; therefore, the deck for Stage 1 should be expected to have a lower response to thermal changes than most concretes. This translates into a reduced risk of cracking in the deck due to thermal effects.

According to the NCHRP 380 cracking potential test for the Stage 1 deck, restrained cracking occurs between approximately 10- and 12-days as shown in Figure 3-6. These results indicate that the concrete used in the Stage 1 deck will crack earlier than concrete from either Test Mix 1 or Test Mix 3, Test Mix 1 would be the best for restrained shrinkage, cracking at 22 to 24-days, while Test Mix 3 is next best at 18-days to cracking. Similarly, the NCHRP 380 results for the Stage 2 deck indicated that restrained cracking occurs between 3 and 11-days.

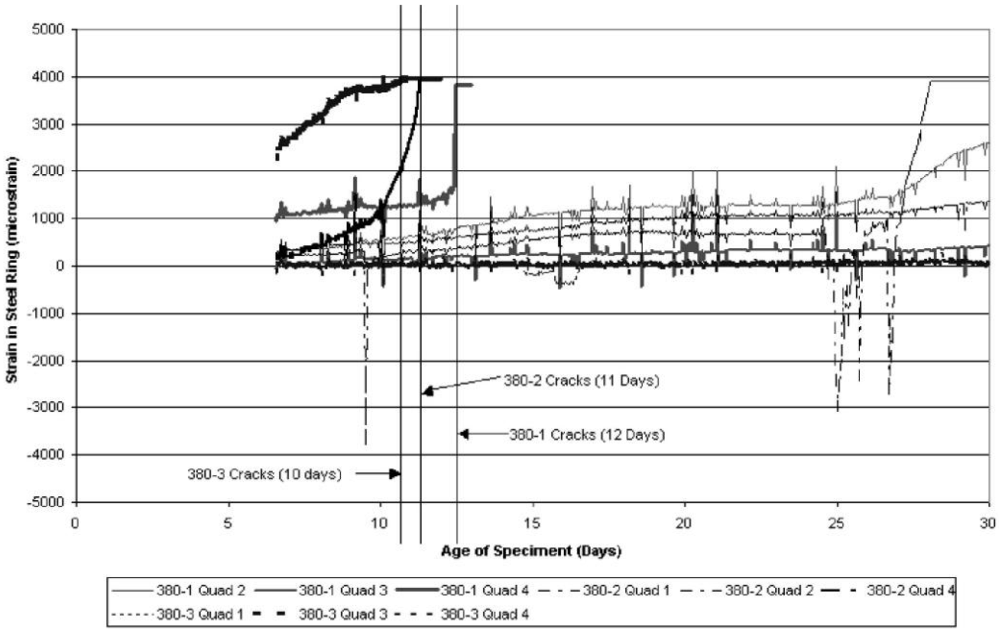

Figure 3-6: Cracking Potential Results for Stage 1 Deck Concrete.

3.2.4 Creep and Shrinkage

The ASTM C512 (creep) results were obtained from loading 3 - 4 x 8 in. cylinders to 20,000 lb (1600 psi) for the duration of the test. The only difference between the Test Mix 1, Stage 1 mix, and the Stage 2 mix were the age at which the specimens were loaded. The cylinders for Test Mix 1 were loaded at an age of 20-days, the specimens for the Stage 1 deck were loaded at an age of 28-days, and the specimens for Stage 2 deck were loaded at age of 13-days. The age at loadings for the Stage 1 and Stage 2 decks roughly corresponds to the date that forms were removed from the deck. The test specimens were accompanied by 3 - 4 x 8 in. control cylinders used to correct the readings on the creep specimens for thermal changes and shrinkage of the concrete. Corrected creep results are shown in Figure 3-7. In this graph and throughout the rest of the paper, tensile strains are positive and compressive strains are negative. At an age of 275-days the total creep, less the strain from the initial loading, had reached approximately -1300 microstrain.

Figure 3-7 shows the results of this test. Comparing the creep of the Stage 1 deck concrete to that of the Stage 2 deck concrete or the concrete from Test Mix 1, it becomes apparent that the Stage 1 deck concrete has a much lower creep rate than the other two mixes. At 28-days after loading, the deck concrete has crept only about -195 microstrain as opposed to -794 microstrain for Test Mix 1.

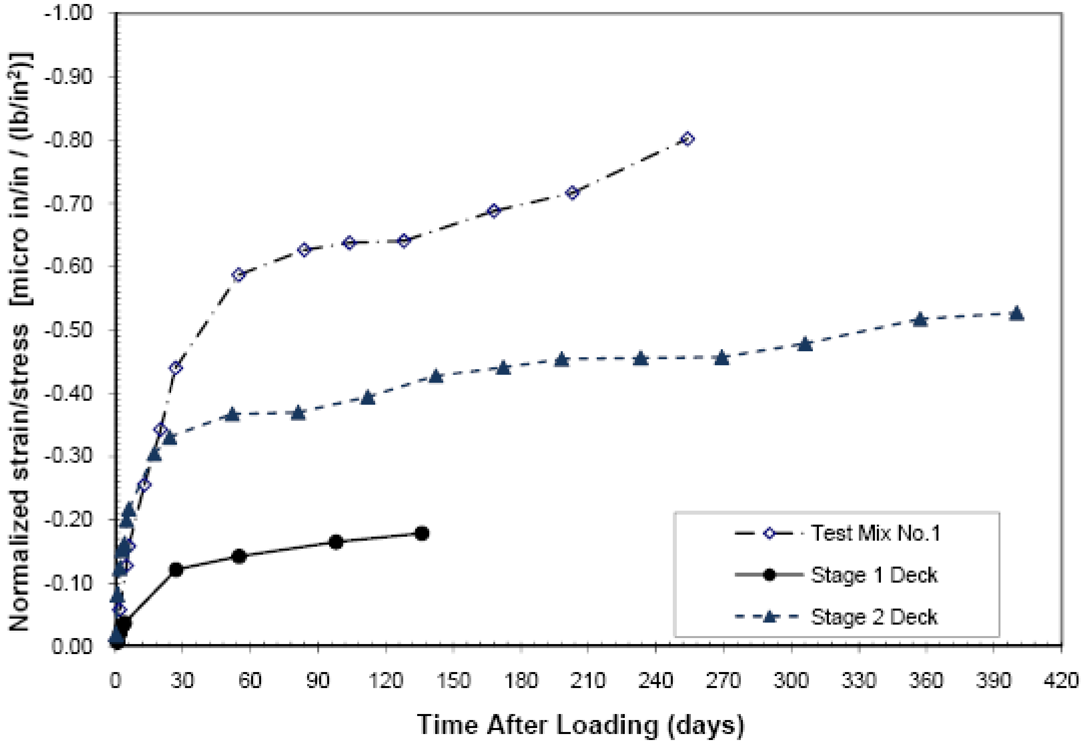

Figure 3-7: Normalized Creep Strains.

Such a low creep value for the Stage 1 deck concrete suggests that it will not dissipate the stresses applied to it as quickly as Test Mix 1 would; therefore the deck concrete has a higher risk of transverse cracking. Readings on the creep in the Stage 1 deck concrete were discontinued at 125-days and had reached a microstrain of about -263. Test Mix 1 had a much higher rate of creep throughout the test and reached about -1360 microstrain at 275-days. In contrast, the Stage 2 deck concrete exhibited a much higher creep rate than the Stage 1 deck concrete. This would suggest that the Stage 2 deck concrete would dissipate stresses more readily than the Stage 1 deck concrete, and that the Stage 2 deck had a lower risk of transverse cracking than the Stage 1 deck.

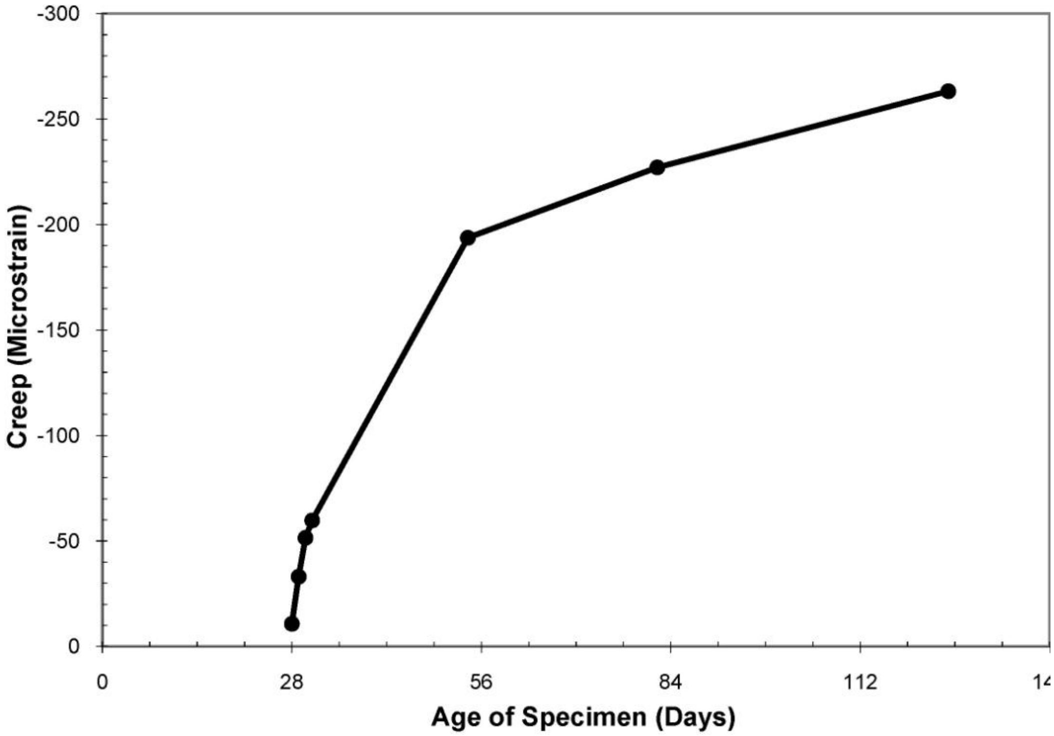

Figure 3-8: Creep Results for Stage 1 Deck Concrete.

The stress that produced the above creep results was about 1600 psi. To get the total creep in the deck at a given time, the graph in Figure 3-7 was normalized by dividing the recorded creep strains by the stress applied to the test specimens. The curve was then shifted to the left so that the creep equation could be used with a start time of one day after pouring. A logarithmic equation was then derived to fit the actual normalized creep data. Shown below is the equation used to estimate creep in the deck.

Equation 3-4: Estimation of Creep in Concrete Deck

$$\varepsilon = (1/E) + F(K)\cdot\ln(t+1),$$

where E is the instantaneous elastic modulus, F(K) is the creep rate, and t is the time after loading, respectively. Total creep is estimated by multiplying the normalized creep by the average stress in the deck. Tensile and compressive creep was assumed to be similar.

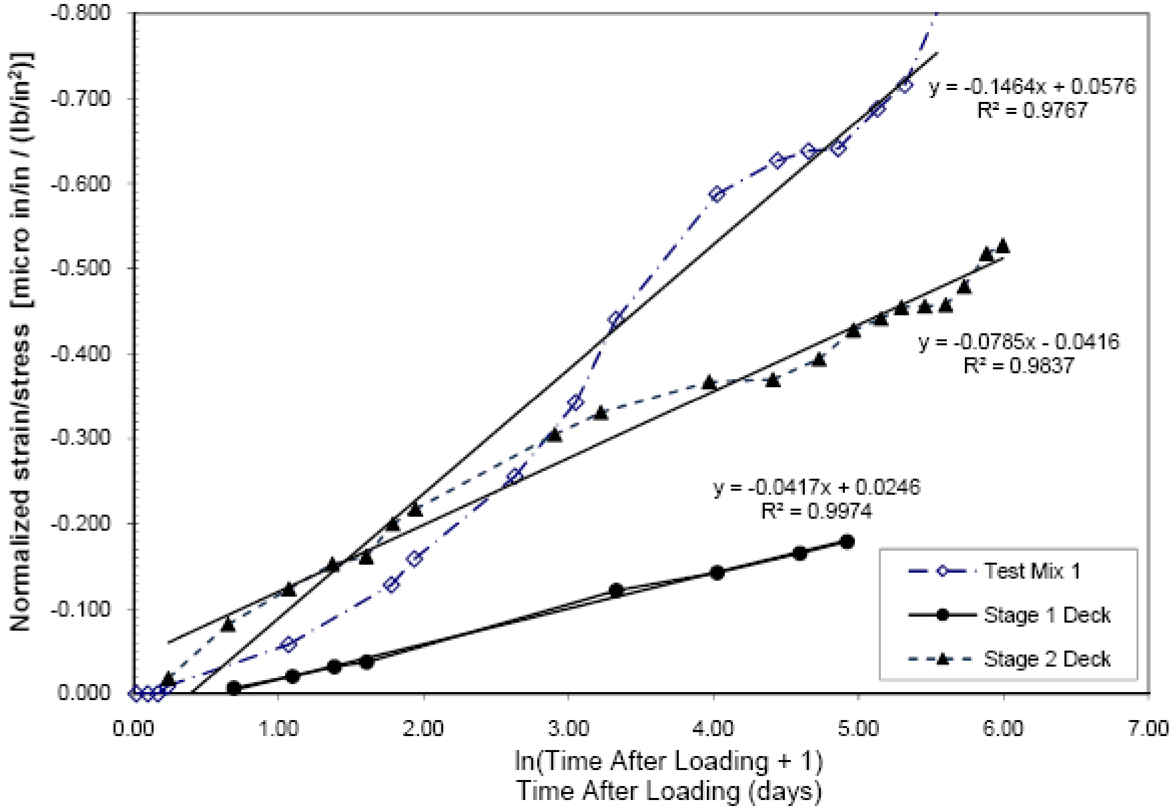

Figure 3-9: Creep Rate for Bridge Deck Concrete.

Drying shrinkage of the deck concrete was also greater than the test mixes. At 28-days, the deck concrete had shrunk 283 microstrain, shown in Figure 3-10, as compared to Test Mix 1 (the worst case of the trial HPC mixes), which only shrunk 100 microstrain. Stresses from drying shrinkage are expected to be higher for the conventional mix then a HPC mix. To determine the drying shrinkage in the deck at a particular time, the equation shown in Figure 3-10 can be used. Since only three data points were taken, it was assumed that a logarithmic equation would best fit the shrinkage data overtime. The curve seems to overestimate the shrinkage at 28-days and underestimate it at 125-days, but it has only about a 1 to 2 percent error at these times and should be adequate for estimating shrinkage in the deck.

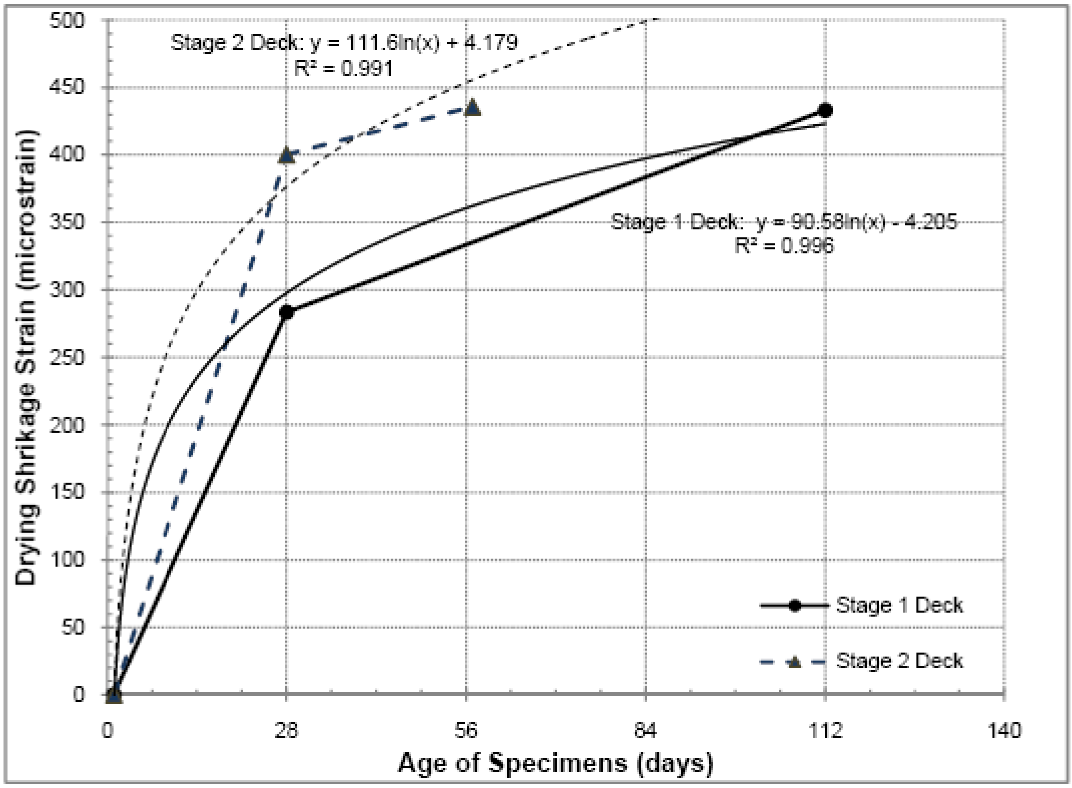

Figure 3-10: Drying Shrinkage Results for Phase 1 Deck Concrete.

The correlation between age of the concrete in days (A) and the drying shrinkage in total microstrain (S_{dry}) is shown in the equations below:

Equation 3-5: Estimation of Drying Shrinkage of Stage 1 Deck Concrete

$$S_{dry} = 90.58 \cdot \ln(A) - 4.205$$

Equation 3-6: Estimation of Drying Shrinkage of Stage 2 Deck Concrete

$$S_{dry} = 111.6 \cdot \ln(A) - 4.179$$

These equations are for the laboratory specimens. Since the surface area to volume ratio is lower for the deck than the test specimen used to derive the above equation, the actual drying shrinkage is less in the deck than the derived equation calculates. To get the actual drying shrinkage in the bridge, the equation needs to be divided by a factor of 2.5[1].

3.2.5 Concrete Durability Tests

As expected, the permeability for the Stage 1 deck concrete was over 4800 coulombs at 90-days. As discussed in Section 3.2.1, the increased permeability compared to Test Mix 3 (2615 coulombs at 28-days) can be attributed to the lack of silica fume and the increased cement content. Although the permeability increases as the concrete ages, the additional age of the deck concrete when tested probably had minimal impact on the overall test results. Test Mix 4 had a higher permeability, but this is most likely from the increased air content and cement content similar to the deck concrete. The permeability for the Stage 2 deck concrete was measured to be 7678 coulombs at 28-days. This value was attributed to the vesicular aggregate used in the concrete.

Abrasion resistance tests on the Stage 1 deck concrete resulted in an average loss of 3.6 grams per 2 minute abrasion period. Similarly, the abrasion resistance tests on the Stage 2 deck concrete resulted in an average loss of 4.2 grams per 2 minute abrasion period. These results indicate that the bridge deck concrete is much more resistant to abrasion than Test Mix 1 (7.3 grams per 2 minute abrasion period) and slightly less resistant to abrasion than Test Mix 3 (3.3 grams per 2 minute abrasion period).

ASTM C672 and C666 tests were performed to determine the scaling resistance and freeze-thaw durability of the deck concrete, respectively. Results of the C672 test indicated that both the Stage 1 deck concrete and the Stage 2 deck concrete had a scaling resistance rating of 1 (very slight scaling with no coarse aggregate showing), which is comparable to the scaling resistance of Test Mix 1. A visual comparison of the tested specimens from each mix confirmed this conclusion.

Although the ASTM C666 test was not performed on any of the test mixes, the test results can be used for comparison between the Stage 1 deck concrete and the Stage 2 deck concrete. As shown in Figure 3-11 and Figure 3-12, the relative dynamic modulus of the bridge deck concrete only decreased by 10 percent after 300 cycles and the specimen only lost approximately 0.82 percent of its mass. According to the agency that performed the test, Construction Technology Laboratories, Inc., the "relative dynamic modulus [of the sample] indicates the specimens are freeze-thaw durable."

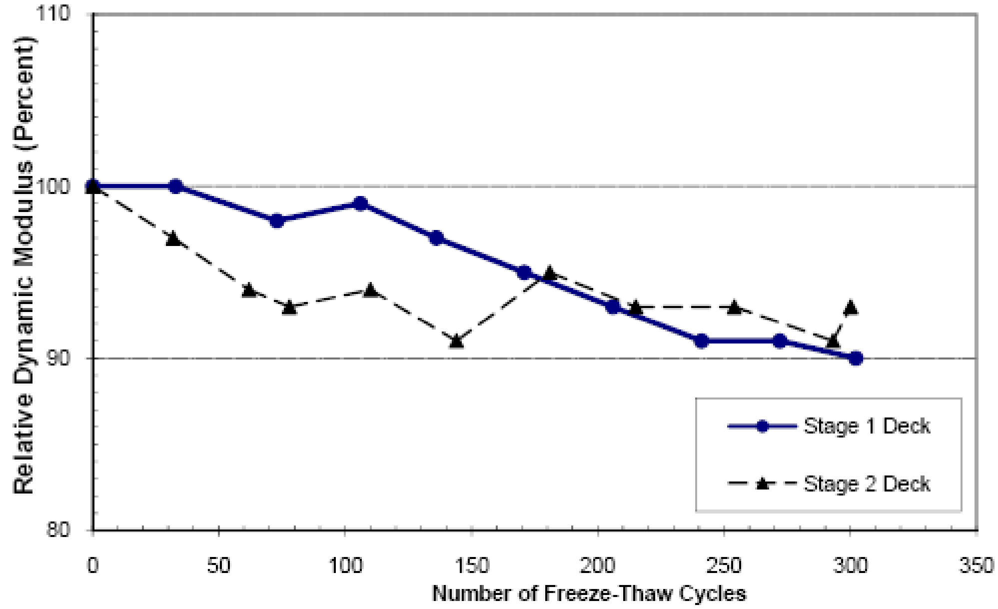

Figure 3-11: Change in Relative Dynamic Modulus with Respect to Number of Freeze-Thaw Cycles (ASTM C666).

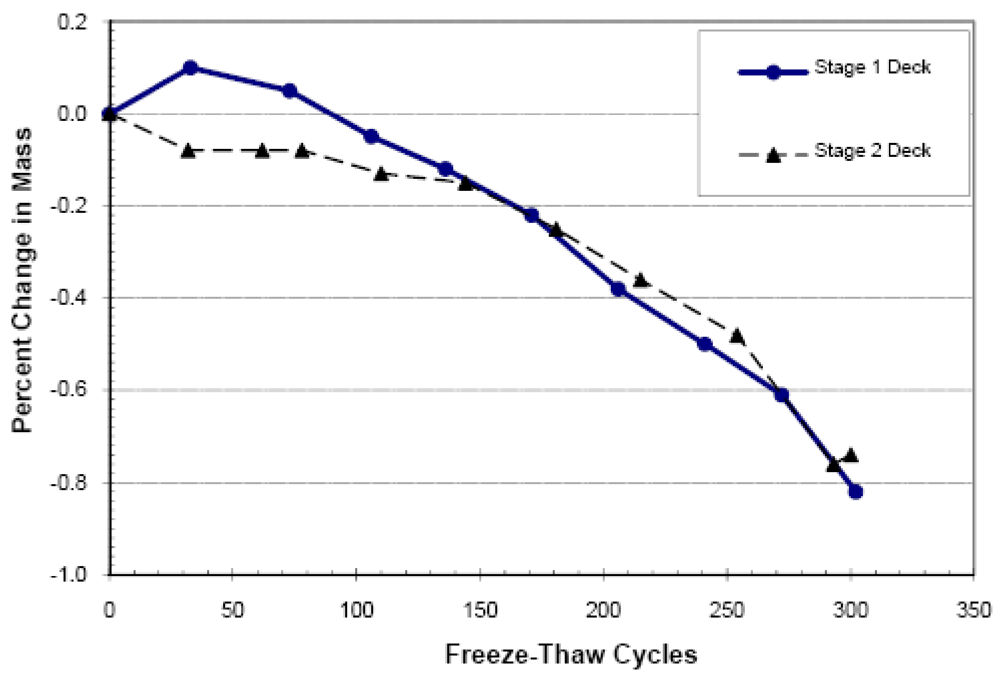

Figure 3-12: Percent Change in Mass with Respect to Freeze-Thaw Cycles (ASTM C666), Freeze-Thaw Durability, Results for Bridge Deck Concrete.

3.3 Summary of Material Test Results

The results of the material tests for both the test mixes, the Stage 1 deck concrete and the Stage 2 deck concrete, indicated that Test Mix 3 best met the specifications as stated by ITD. Both the cement content and permeability are much lower than any of the other test mixes or Stage 1 deck concrete. In addition, the abrasion resistance of Test Mix 3 is lower than any of the mixes tested. Although cracking occurred sooner than either Test Mix 1 or the deck concrete in the NCHRP 380 test, the mix design properties indicate that Test Mix 3 would be more durable and have a lower risk of transverse cracking. Batching of Test Mix 3 was not performed by the concrete company that won the bid for this project so it could not be used on the bridge deck.

Additional tests were performed on Test Mix 1, the Stage 1 deck and the Stage 2 deck concrete. Creep test results for the these tests revealed that Test Mix 1 has much higher creep than the both the deck Stage 1 deck concrete and the Stage 2 deck concrete and could therefore sustain greater strains before cracking occurs. The drying shrinkage of Test Mix 1 was also considerably lower increasing the time before cracking would occur. This was verified by the results from the cracking potential tests. The Stage 1 deck concrete had a lower coefficient of thermal expansion, however, so it would be affected by thermal changes less than Test Mix 1. The tensile strength and abrasion resistance were also higher. The durability of the Stage 1 deck concrete and the Stage 2 deck concrete seems to be adequate.

The correlations and results of the material tests for the deck concrete of the Stage 1 deck and the Stage 2 deck are used later in this report to determine the stresses in the bridge deck based on the recorded strains. The portion of the stresses caused by shrinkage and thermal changes and reduced by creep is also presented. In addition, the locations of actual transverse cracking in the deck presented and discussed.

DECK CONSTRUCTION DETAILS AND OBSERVATIONS

Placement, curing, and finishing of the bridge deck can have a significant effect on plastic shrinkage, settlement cracking, plastic flexural cracking, proper hydration of the cement, and hydration temperatures. Section 4.1 discusses observations made on construction methods for the bridge deck pour followed by a discussion of the design details of the bridge in Section 4.2. As shown in Illustration 4-1, the bridge deck between Girders #5, #6, and #7 was placed in the Stage 1 of the construction. The bridge deck between

Girders #1, #2, #3 and #4 was placed during Stage 2 of construction. A closure pour between Girders #4 and #5 was used to complete the bridge deck.

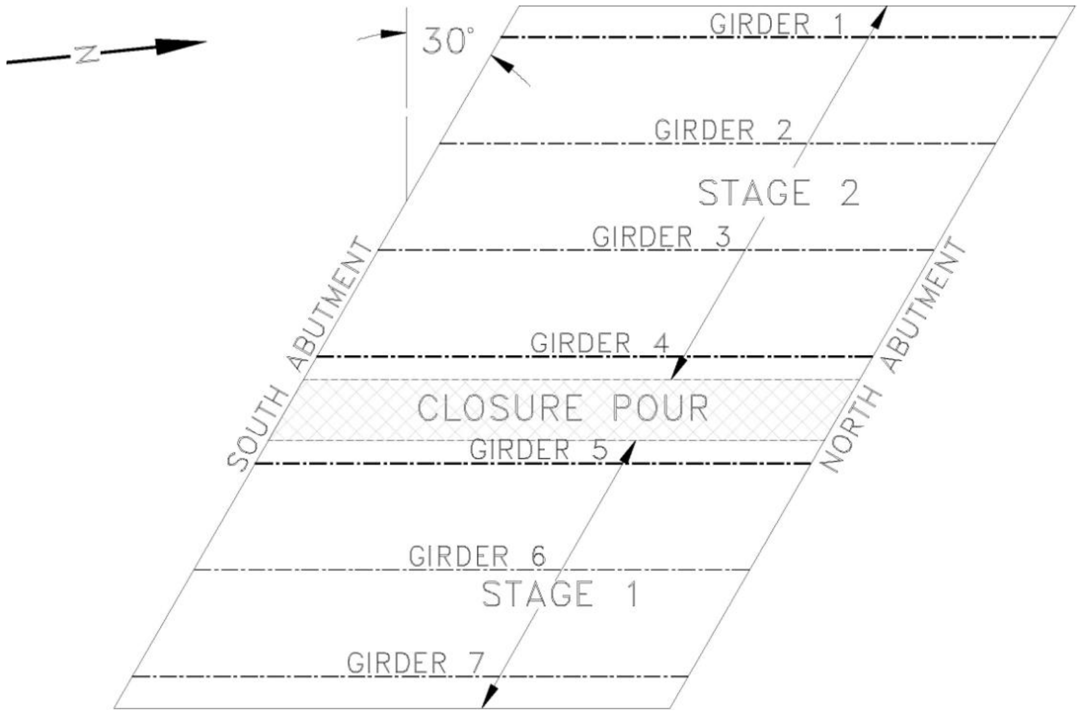

Illustration 4-1: US 95 Bridge - Deck Construction Stages.

4.1 Construction Observations

Sections 4.1.1 through 4.1.4 present the observations made on the construction procedures used for first stage of the bridge deck pour. Section 4.1.5 then compares the observations made to the suggestions of the literature review.

4.1.1 Deck Pour Preparation

The formwork for the bridge deck, shown in Illustration 4-2, consisted of ¾ in. thick plywood supported on 2 x 4 in. boards at 12 in. centers running perpendicular to the girders. The 2 x 4's were supported by 2 laminated 2 x 10 in. beams running parallel to the girders and supported by metal truss pieces attached to the side of the girders.

Illustration 4-2: Phase 1 Bridge Deck Formwork.

Prior to deck placement, an enclosure was constructed below the formwork. A plastic side curtain was attached to the outside edges of the forms to provide insulation so that propane heaters could heat the enclosure to approximately 50° F. Heating of the main enclosure was ceased on 11/12/03 and the enclosure was removed on 11/28/03. This provided a gradual decrease in deck temperature to ambient air temperature as discussed in Section 1.3.1.

4.1.2 Concrete Placement

Concrete was placed on the deck with a 2 yd^3 crane-suspended bucket. Hand shovels were used to spread the concrete out after discharge from the bucket. Pouring was done from the North side of the bridge to the South side in a continuous manner. Placement started on 11/07/03 at 9:00 a.m. on the North abutment. Air Temperature at this time was 38° F and increased to 40° F by the end of placement. At 10:15 a.m. the abutment placement was finished and deck placement on the North side of the bridge commenced. By 12:00 p.m. the North half of the bridge had been placed and finished. Placement of the deck continued until 12:45 p.m. At that time, placement of the South abutment started and continued until 1:45 p.m. The remaining deck concrete was then placed. The final concrete placement was made at 2:30 p.m. and finishing of the deck was done by 3:15 p.m. Total pour time was about 5½ hours, 3 hours of which constituted deck placement.

The concrete was batched using heated water and arrived at the site at a temperature of about 60º F. Total time from batching to the start of discharge from the truck averaged about 23 minutes and took no more than 32 minutes and no less than 15 minutes. Total discharge time took about 21 minutes with a maximum discharge time of 30 minutes and a minimum of 10 minutes.

4.1.3 Finishing Procedures

The deck was initially consolidated using 1 hand-held vibrator followed by a Bidwell finishing machine, shown in Illustration 4-3, to get a uniform deck thickness across the deck. A vibrating screed on the Bidwell machine performed additional consolidation of the concrete. Concrete near the edges of the deck was finished by hand.

Illustration 4-3: Finishing Equipment for Deck Concrete.

4.1.4 Curing Procedures

After final finishing of the deck concrete, an evaporation retarder film was applied when bleeding of the concrete ceased. At 1:30 p.m. the North half of the bridge deck was sprayed, approximately 1½ to 3 hours after finishing. The South half was sprayed at about 3:30 p.m., approximately 45 minutes to 3½ hours after finishing. When the entire deck had been sprayed with the evaporation retarder film and had been allowed to set for at least an hour, around 4:00 p.m., the deck was covered with a layer of plastic to hold in moisture followed by a layer of black geofabric to serve as insulation. The insulation was removed from the deck 10 days later on 11/17/03. On 11/12/03 the parapet wall and sidewalk was poured and an enclosure was built around them and heated.

4.1.5 Comparison of Construction Procedures to Literature

Comparing the observed construction procedures to the suggested practices discussed in Section 1.3.1 reveals that most suggested construction procedures were followed on the deck pour. This was a single-simple-span deck pour that was done in one placement as suggested by the literature review; however, it may have been better for restraint and cracking considerations if each abutment was poured up to the level of the bottom of the deck followed by pouring the entire deck in one continuous pour. Instead, the North half of the bridge was poured starting with the abutment and continuing into the deck until the deck concrete was within about 10 ft of the South abutment. The fresh deck concrete was then left for over an hour while the South abutment was being poured. During this time, the deck concrete was setting. When placement resumed on the deck, it was not against fresh concrete, creating a very minor construction joint in this location and an increased chance of cracking.

The contractor followed the suggested cold weather concreting procedures outlined in Section 1.3.3 and in the project specifications. As suggested, warm water was used in the concrete mix to keep the concrete temperature at the time of placement between the suggested temperatures of 55° F to 75° F. The girders and underside of the forms were also heated to reduce temperatures differentials between them and the deck. In addition, the deck was poured on a sunny day and then insulated from the cold to keep deck temperatures up.

Proper consolidation of the concrete was also performed. Although 1 one hand-held vibrator was used, the placement rate was maintained at approximately 20 yd^3/hr, which is below the suggested 30 yd^3/hr limit. The vibrating screed on the Bidwell machine also helped with consolidation, and as expected, no settlement cracking was observed on the deck. Prior to placement, the forms and reinforcement were wetted down with water as suggested. In addition, other suggestions from the literature for the finishing and curing procedures were also met. An evaporation retarder film was used to reduce evaporation. After placement the deck was insulated and a vinyl cover was used to also reduce evaporation. Section 5 will further discuss the evaporation rate prior to application of the retarder film, but no plastic shrinkage cracks were observed on the deck so evaporation was not a problem.

4.2 Design Details

The overall bridge design can greatly affect the overall restraint on the bridge. Section 4.2.1 looks at the restraint caused by the girders followed by any additional restraint from shear connectors and abutments in Section 4.2.2. Section 4.2.3 looks at any other design details and compares them to suggestions in the literature review.

4.2.1 Girder Type, Size, and Spacing

According to the suggestions given in the literature review, the prestressed-precast concrete girders used for this bridge are one of the best types of girder to reduce transverse cracking. Girders cast-in-place with the deck are most effective at reducing transverse deck cracking, but have other disadvantages. Since the shrinkage and creep in the girders is similar to that in the bridge deck, the total stress in the bridge deck is reduced. The girders for the bridge were poured approximately 2 months before the Stage 1 deck was placed. At this time, the majority of the curing had already occurred in the girders, but they still continue to shrink and creep. The girder dimensions, bridge cross section, and closure pour details are shown in Illustration 4-4, Illustration 4-5, and Illustration 4-6. The girders were 45 in. AASHTO Type III Girders spaced at 12 ft 6 in. centers. The spacing is about twice the normal spacing used on bridges of this type and should reduce the restraint on the deck from the girders. Because of the increased spacing, however, the girders are larger than usual and so the benefits are partially offset.

4.2.2 Additional Restraint

So that expansion joints would not need to be constructed, the girders were cast integrally with the abutments. For larger stresses such as those imposed by larger service loads and the maximum design loads, the integral abutments act as pinned connections; however, for lower level stresses such as those from thermal changes and shrinkage, the abutments and sidewalk details (Illustration 4-7) effectively act as fixed-fixed supports. This restrains longitudinal shrinkage or expansion of the girders. The additional restraint effectively negates any benefits from shrinkage or creep provided by the precast concrete girder because the girders do not shrink with the deck. This increases the stresses in the deck from shrinkage or thermal changes and increases the risk of transverse cracking. Decreasing the restraint on the girders by not pouring them integral with the abutments may decrease the risk of transverse cracking considerably.

The shear connectors on top of the girder add to deck restraint; although they do not restrain the deck as significantly as the abutments. Created using the shear stirrups in the girder, as shown in Illustration 4-3 (note: dimensions are in millimeters), the shear connectors are approximately 5 in. tall and about 5 in. apart in the transverse direction. Longitudinal spacing and size of the stirrups varies from No. 5 bars spaced at 6 in. near the end to No. 4 bars spaced at 20 inches in the middle. The top of the girder is roughened to aid in transferring shear from the deck to the girders.

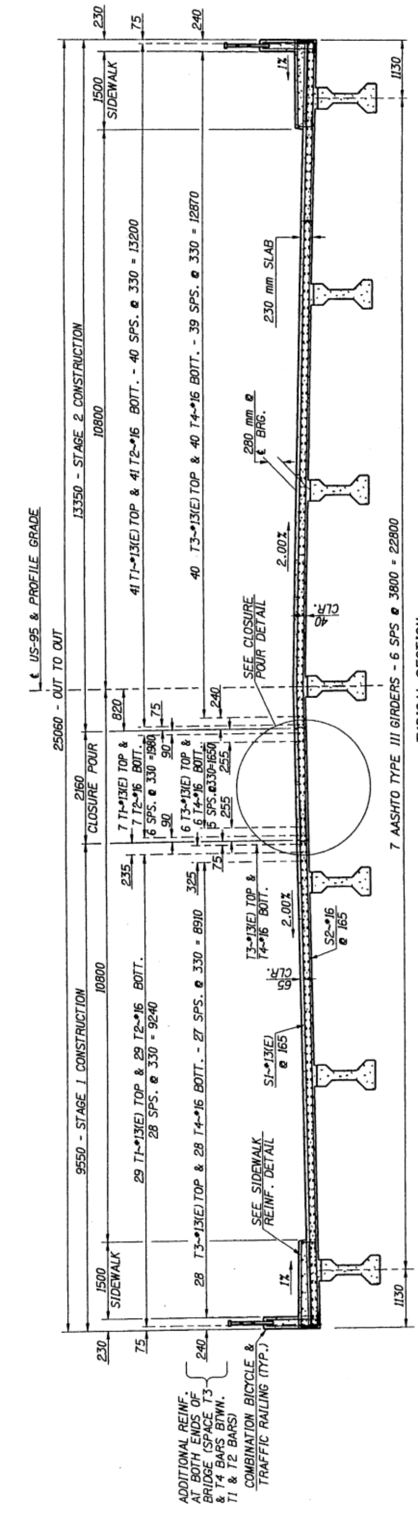

Illustration 4-4: US 95 Bridge - Typical Section[17].

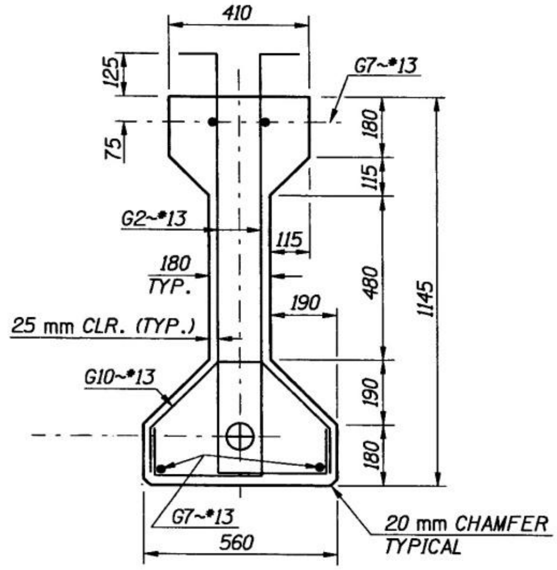

Illustration 4-5: US 95 Bridge - Cross Section of Prestress-Precast Girder[17].

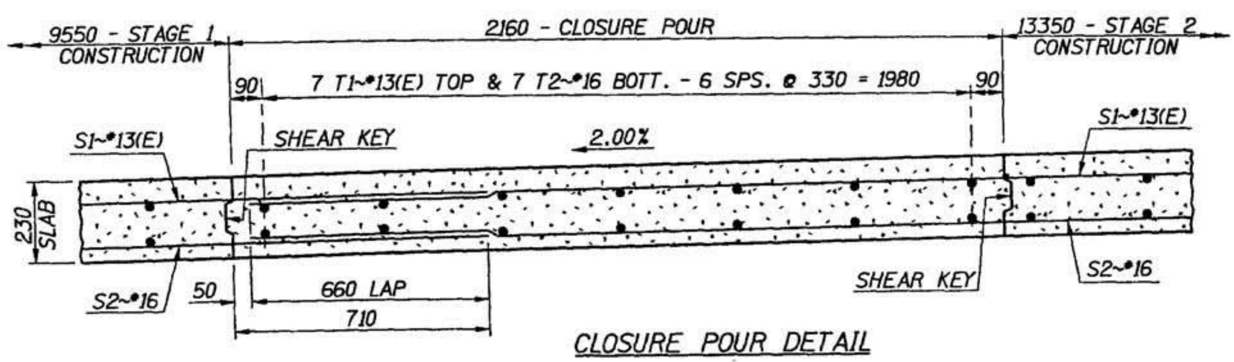

Illustration 4-6: US 95 Bridge - Closure Pour Detail[17].

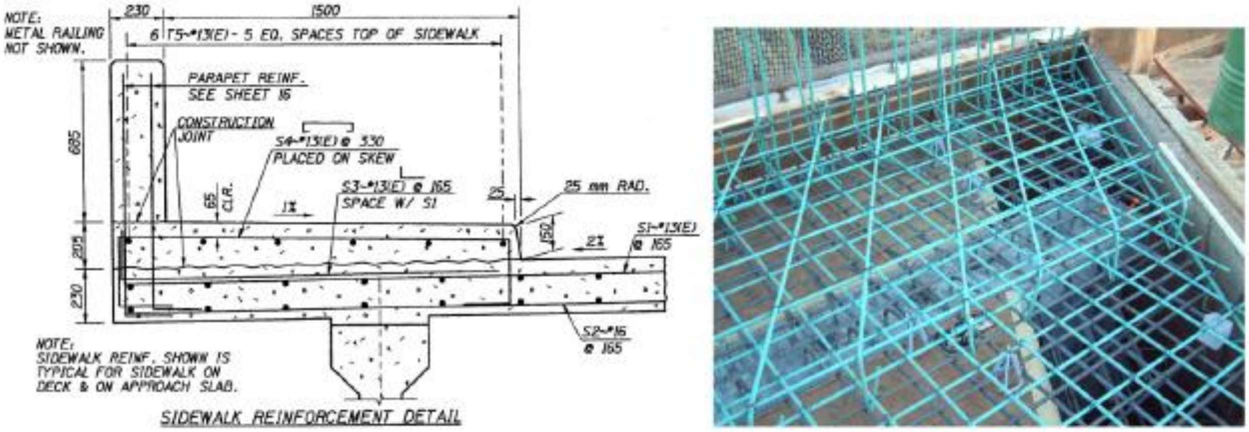

Illustration A-1~~7~~: US 95 Bridge-Sidewalk Reinforcement Details[17].

4.2.3 Other Design Details

Other design details of the South Fork of the Palouse River Bridge are shown in Table 4-1 along with suggested values from the literature review.

Table 4-1: Comparison of Observed Design Details to Suggested Values

Design Item	Observed Value	Suggested Value
Span Length	~ 64 ft	Less than 90 ft
Deck Width	Stage 1: ~31 ft Stage 2: ~ 35½ ft Total: ~82 ft	Less than 70 ft
Skew	30°	Less than 30°
Reinforcement Size	Top: No. 4 Bottom: No. 5	No. 5 or smaller
Reinforcement Type	Top: Epoxy Coated Bottom: Black Bar	Epoxy coated bars increase cracking but reduce deterioration from deicing chemicals. (Designers choice)
Longitudinal Spacing	End: 6½ in. (Extends 9½ ft into deck) Middle: 13 in.	Less than 6 in.
Transverse Spacing	6½ in.	Less than 6 in.
Alignment of Top and Bottom Reinforcement	Top and bottom aligned and transverse bars above longitudinal bars	Avoid alignment and put longitudinal bars above transverse bars
Top Clear Cover	2½ in.	Between 1½ to 3 in. (preferably 2 in.)
Deck Thickness	9 in.	Greater than 8½ in.

Span length, reinforcement size and type, top clear cover, and deck thickness are all within the limits suggested by the literature review. Transverse spacing of the reinforcement and longitudinal spacing near the end of the deck are within ½ inch of the suggested values. The longitudinal spacing in the middle of the deck, while more than double the suggested limit, is within the limits required by the AASHTO LRFD Bridge Design Manual[19]. Closer longitudinal spacing helps distribute longitudinal tensile stresses in the deck. The effect on transverse cracking is minimal, so this design detail probably did not increase the risk of transverse cracking.

The alignment of the top and bottom reinforcement is a much more significant design factor. Since the top and bottom reinforcement is aligned in the bridge deck being studied, there is a plane of weakness located at each reinforcement location, increasing the risk of cracking. The literature review states that most transverse cracking occurs directly above the top reinforcement and placing the transverse bars above the longitudinal bars increases this risk of cracking. The increased risk is from the smaller effective area of concrete that can resist the longitudinal stresses.

The deck width is another design factor that exceeded the suggested value. Individually, the Stage 1 and Stage 2 deck width do not exceed the 70 ft limit, but the entire bridge deck width does. A wider deck increases the amount of concrete that can shrink. Since the abutments restrain the shrinkage of the deck in both the longitudinal and transverse direction, the wider deck can increase the tensile stresses in the top of the deck when it shrinks to well above rupture strength, increasing the risk of transverse cracking. However, since the deck was constructed in two stages, with several months separating the placing of the concrete for the two stages, the decks act independently for early age cracking effects.

Finally, the skew of the deck for the bridge in review is at the suggested limit. The greater skew of the deck increases the uplift that occurs at the acute angles of the deck. For this deck, the uplift is restrained by the abutments and girders and also by the parapet wall and sidewalk near the corner. Illustration 4-8 shows the cracking that had occurred in the bridge deck up to 4/15/04 (a deck age of 160-days). The adverse effects of skew are apparent by looking at the South East corner of the deck. The cracks at 5 ft 1 in., 7 ft 1 in., and 10 ft 1 in. from the end are all caused by uplift in the deck and the restraint from the parapet wall, sidewalk, and abutments. Further analysis of the cracking shown is presented in Section 5.

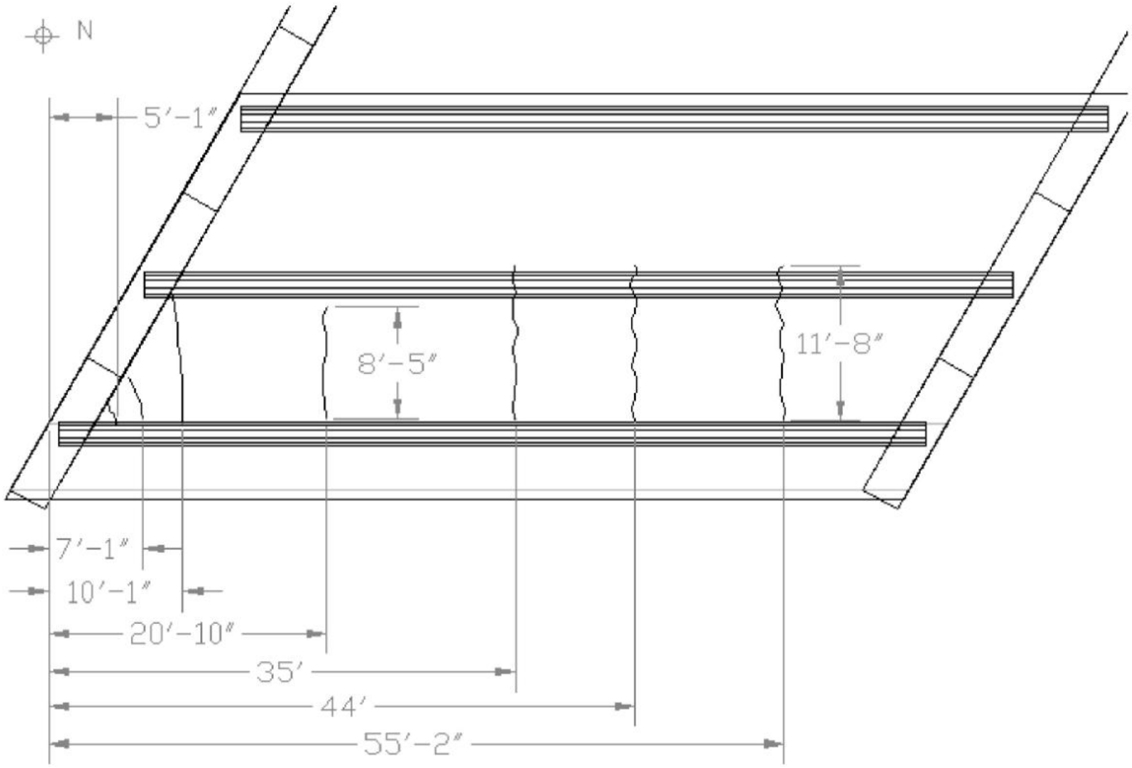

Illustration 4-8: Cracking in the Stage 1 Bridge Deck at 160-Days.

DECK MONITORING AND INSTRUMENTATION

The South Fork of the Palouse River Bridge was instrumented with strain gauges and thermocouples as discussed in Section 2.2. The weather near the site was also monitored. For comparison purposes the instrumentation-monitoring period was divided into two time intervals. The first interval is from deck placement (11/07/03) to 21-days after placement (11/28/03). During this time, the concrete was in its early strength gain period and significant temperatures increases were recorded resulting from the heat of hydration, insulation and under-deck heating. Temperatures and strains were rapidly changing and better resolution was needed during this time to notice trends in the data. On 11/28/03 the formwork was removed from the bottom of the deck and drying shrinkage of the bottom of the deck effectively started. The second interval was after the insulation and enclosure was removed (on 11/28/03). Weather was then the dominant factor in drying shrinkage and thermal effects on the bridge.

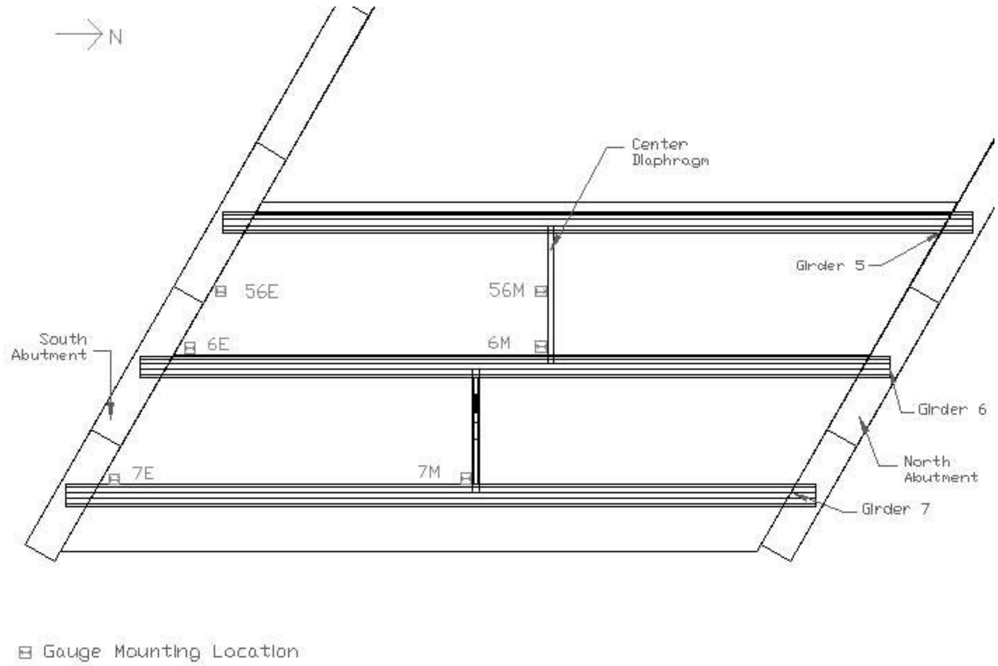

Illustration 5-1: Gauge Location Nomenclature.

Data associated with an individual gauge is identified the same way on all figures throughout Section 5 and the Appendices. Illustration 5-1 shows the location of all gauge configurations and the primary code identifying the configuration. Each configuration is identified by one or two numbers designating the number or numbers of the closest girders followed by either an M or E to designate a configuration located near the middle or end of the girder, respectively. Individual gauges, within a configuration, are further identified by their mounting type, where E, B, and G represent embedment, bottom mount, and girder mount, respectively. For example, the bottom mount gauge near the end of Girder 6 is designated as gauge 6E-B.

Analysis of the instrumentation data was broken up into four sections. The first section uses data from the UI Plant Science weather station to look at the evaporation rate during placement of the deck. After placement is complete, the only weather data that has a significant effect on the bridge deck is the air temperature. It is presented in Section 5.2 along with deck temperatures. Section 5.3 analyzes the strain data that was recorded as well as calculates the expected strain using temperature data and material test data. Section 5.4 uses the difference between the calculated strain and the actual strain to determine the stress in the deck.

5.1 Evaporation during Concrete Placement

The weather at the time of casting affects the risk of plastic shrinkage cracking in the deck, especially the evaporation rate. The weather station for the UI Plant Science Farm near the project site was used to collect data on solar radiation, air temperature, relative humidity, wind speed, and precipitation. Although the station was approximately two miles from the project site, conditions were similar at both locations. Wind speed may be slightly less above the deck due to wind blocks created by concrete barriers and sheet piles near the bridge. During placement, when wind speed is most important, the deck had small wind blocks, created from the formwork.

To determine the evaporation rate while the deck was being poured for Stage 1, Figure 1-1 was used along with the weather data recorded at the UI Plant Science weather station. During this time, the relative humidity and concrete temperature remained fairly constant at 33 percent and 50° F, respectively. Although the air temperature varied between 38° F and 42° F, the evaporation rate was not affected significantly, the evaporation rate nomograph values change very little for a given wind speed below 45° F. Wind speed was the only weather variable that had a significant effect on the evaporation rate. As shown in Figure 5-1, the maximum evaporation rate suggested in section 1.3.1 for concrete with water-to-cement ratio less than 0.40 (0.10 lb/ft^2/hr) was not exceeded. As expected, plastic shrinkage cracking was not observed on the deck.

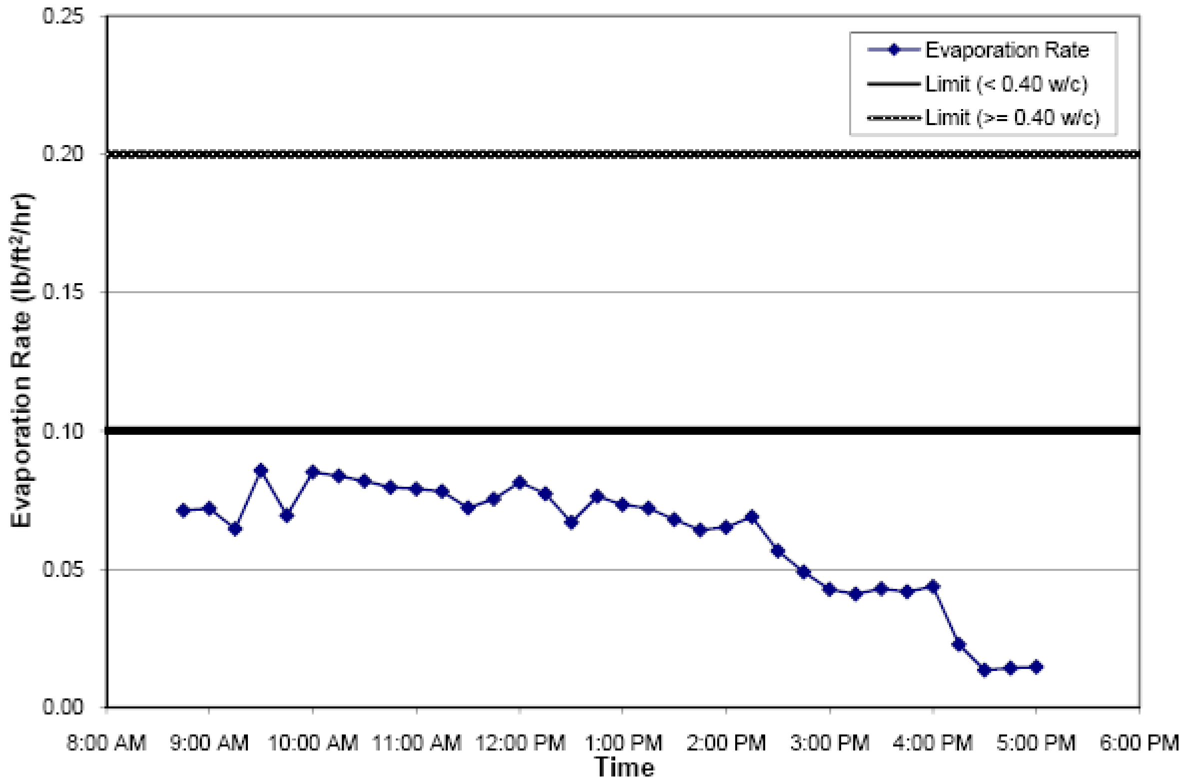

Figure 5-1: Evaporation Rate During Stage 1 Deck Pour.

5.2 Deck and Air Temperatures

Graphs of the deck temperatures at each gauge configuration are shown in Appendix A. The air temperature, based on the UI Plant Science weather station data, is also shown on each graph for comparison purposes. As discussed earlier, each graph was broken up into two time intervals. During the first time interval, heat from hydration of the cement had the greatest effect on deck temperature. The temperature of the bridge enclosure and insulation on the deck had an additional effect on the deck temperature. As discussed in Section 4.1.1, the contractor continued to heat the enclosure below the deck for 10-days after placement. When the peak temperatures in the deck were reached, shown in Table 5-1, the air temperature directly below the formwork was about 85° F, which is 10° F warmer than the suggested limit presented in Section 1.3.3 for placement temperatures. The increased enclosure temperature increased the overall hydration temperature in the deck and thus the thermal strains. To help reduce maximum hydration temperatures, heating of the enclosure should gradually be decreased starting immediately after placement of the concrete so that the 75° F limit given for placement of the concrete is maintained while the cement is in the initial hydration phase.

In addition, the enclosure had other effects on the deck temperature after the maximum hydration temperature was reached. On 11/8/03 around 11:30 a.m., the insulation on the deck and plastic enclosure cover near gauge configuration 7E was removed. Figure A-3 shows the resulting drop in temperature. The embedment gauge temperature dropped about 18° F and the girder mount gauge temperature dropped about 10° F. The bottom mount gauge was unaffected. This was probably due to the insulation that the formwork provided on the bottom of the deck. On 11/11/03 at about 8:00 a.m., all of the bottom mount and girder mount gauges started to dramatically drop in temperature. This was due to the enclosure being opened up so that the parapet wall enclosure could be constructed, dropping the bottom of the deck and the girder temperatures around 20° F. The embedment gauges were unaffected.

The enclosure was closed back up at the end of the day (4:00 p.m.) and the temperature in the enclosure rose to normal levels. While the parapet wall and sidewalk were being poured (11/12/03), the enclosure was again opened up and the bottom and girder mount gauges again dropped in temperature but only about 2° F to 5° F. Closing of the enclosure brought temperatures back to normal; however, pouring the parapet wall and sidewalk had a significant effect on the temperature of the embedment gauges at gauge locations 7M and 7E (exterior girder below the sidewalk). Figure A-1 and A-3 shows the increase in temperature of these gauges. Apparently, hydration of the parapet wall and sidewalk generated enough heat to raise the temperature of the deck. The embedment gauges at location 7M rose about 4° F and 7E rose about 6° F.

Hydration of the deck concrete increased temperatures dramatically. Table 5-1 shows the maximum temperatures at each gauge location, the difference in the temperature of the concrete at the time of pour compared to the maximum temperature, and the time it took for the concrete to reach the maximum temperature. The maximum temperature recorded was at the end of the deck between Girders 5 and 6 at 124.5°F, almost 56° F warmer than when the concrete was poured. The increase at this location is due to the large volumes of concrete in the abutments at the ends of the bridge. The reduced temperatures at the abutments compared to gauge location 56E is due to the girders near these gauges acting as a heat sink, reducing the maximum temperature in the deck near the girders. Temperatures were also lower near the exterior girders because heat was allowed to dissipate more quickly from the outside edge of the deck. In general, during the hydration period, embedment gauges reached the highest temperatures followed by bottom mount gauges. As expected, the girders had the lowest temperatures because they were already hydrated at the time of deck pour. Their temperatures did increase considerably, indicating that they conducted heat from the hydrating deck.

Table 5-1: Maximum Hydration Temperatures

Gauge ID	Maximum Temp. (°F)	Pour to Max Temp. Difference (°F)	Time from Pour to Max. Temp (hr)
7M-E	99.5	31.69	23.5
7M-B	101.4	45.03	48
7M-G	98.2	45.61	48
7E-E	101.4	35.67	16.5
7E-B	99.7	42.37	46
7E-G	85.2	30.65	16
6M-E	109.8	44.04	34.5
6M-B	103.1	42.32	48.5
6M-G	99.3	42.18	48.5
6E-E	115.1	51.51	46
6E-B	104.5	44.28	46
6E-G	98.9	41.47	46
56M-E	112.0	45.97	30
56M-B	100.9	39.46	34.5
56E-E	124.5	55.84	35.5
56E-B	Gauge Detached	Gauge Detached	Gauge Detached

It took the deck between 23.5 hours and 48.5 hours to reach maximum temperatures. Gauges 7E-E and 7E-G reached maximum temperatures about 16 hours after deck placement, but as discussed earlier, removal of insulation at this location caused the deck to not reach its maximum expected temperature at the time that was expected based on the other gauges. In general, the embedment gauges and those gauges located between the girders reached their maximum hydration temperature more quickly than other gauges these were also the gauges with the highest maximum hydration temperatures. This would suggest that the maximum temperatures are caused by shorter hydration times as expected; however, the location with the highest temperature does not have the shortest time to maximum. This indicates that other factors such as insulation or conduction from girders has an effect on time to maximum.

As presented in Section 1.3.1, the literature suggests that the temperature differential between the girder and the deck during the first 24 hours after placement should be kept below 22° F. For this bridge, the deck/girder temperature differential during the first 24 hours after placement was as follows: gauge location 7M was 15° F, gauge location 7E was 10° F, gauge location 6M was 18° F, and gauge location 6E was 23° F. Only gauge location 6E exceeds the suggested limit; however there was no cracking at this location. Gauge locations 7M, 7E, and 6M all had cracks near them that were primarily caused by thermal strains, as discussed in Section 5.3.2, but they did not approach the suggested limit. This suggests that the integral abutments provide enough restraint on the girders for thermal changes, that the Section 1.3.1 suggested temperature differential limit is invalid for integral abutment designs. The maximum hydration temperature in the deck has a greater effect on the thermal strains in the deck then the girder/deck temperature differential.

After 11/28/03, the enclosure and insulation on the deck had all been removed and weather was then the dominating factor in deck temperatures. On 1/5/04 at 1:15 a.m., the lowest temperature that the bridge deck had yet experienced was recorded at -19.6° F. Table 5-2 shows the minimum temperature that each gauge recorded and the time delay between the minimum air temperature and the minimum gauge temperature. Also shown are the differences between the minimum air temperature and minimum gauge temperature as well as the maximum and minimum temperatures at each gauge.

The minimum temperature in the bridge deck was recorded at gauge location 56M-B. It was recorded about three hours after the minimum air temperature was recorded and was about 11°F warmer than the minimum recorded air temperature. In contrast, the warmest minimum temperature was recorded at gauge 7E-E. Almost 33°F warmer than the minimum recorded air temperature, the gauge recorded its minimum 34 hours after the minimum air temperature was recorded. The reason for such a dramatic difference in minimum temperatures and delay between the minimum air temperature and minimum gauge temperature at these locations is due to the differences in the volume of concrete. At location 7E, the deck, abutment, girder, parapet wall, and sidewalk all provide concrete that act as heat sinks. Location 56M only has the deck concrete to retain heat from when the air temperature was warmer.

Table 5-2: Minimum Recorded Temperatures

Gauge ID	Minimum Temp. (°F)	Time Difference from Minimum Air to Gauge Temp (hr)	Difference Between Min. Air and Gauge Temps. (°F)	Difference Between Max. and Min. Temp. at Gauge (°F)
7M-E	11.26	3	30.87	88.24
7M-B	1.37	5	20.97	100.03
7M-G	0.97	5	20.58	97.23
7E-E	12.86	3	32.47	88.54
7E-B	-0.56	3	19.05	100.26
7E-G	-3.04	3	16.56	88.24
6M-E	-5.63	3	13.98	115.43
6M-B	-5.63	3	13.98	108.73
6M-G	-4.44	3	15.17	103.74
6E-E	-0.93	7	18.68	116.03
6E-B	-2.09	4	17.52	106.59
6E-G	1.31	4	20.91	97.59
56M-E	-3.28	8	16.33	115.28
56M-B	-8.38	3	11.23	109.28
56E-E	-0.96	8	18.64	125.46
56E-B	-4.26	3	15.35	108.76

The difference between the maximum and the minimum gauge temperature is what produces the majority of the tensile strains from thermal changes. The largest difference occurs at gauge location 56E-E, which is also the location of the maximum deck temperature. In this case, the girders provide a heat sink while the deck is hydrating as well as during diurnal temperature changes. Since 56E does not have a girder near it, it has a higher maximum temperature and a lower minimum temperature, producing the large temperature difference between maximum and minimum deck temperatures. The temperature at gauge 56M-E is slightly lower because the volume of hydrating concrete near it is much less, reducing the maximum temperature.

5.3 Deck Strains

The strains at each gauge were recorded along with the temperatures; however, the overall recorded strain reading does not by itself determine the stresses at that location. Sections 5.3.1 to 5.3.2 discuss how the recorded strain data was processed to produce data that is relevant in calculating the deck stresses. Section 5.3.3 then discusses the calculation of the restraint on the deck.

5.3.1 Gauge Initialization

When the deck concrete is poured, hydration causes the concrete temperature to increase. The increase in temperature in turn causes the concrete to expand; however, the modulus of elasticity is extremely low during this time. The concrete therefore expands without any increase in compressive stress. As the concrete hydrates and approaches its peak hydration temperature, the concrete begins to “set”, increasing the modulus of elasticity and the amount of stress that develops in the deck. Although the gauges began recording the strain from the time of placement, it is not until the rate of temperature change from hydration begins to slow that stress buildup occurs. For all the bottom mount and embedment gauges this occurs at approximately 5:00 a.m. on 11/08/03, about 17 hours after placement. Stresses in the girders could theoretically occur at anytime because the concrete was precast; however, the abutments do not provide any restraint until the abutment concrete sets. Figure 5-2 shows the overall change in strain from the time of placement of the concrete. Figure 5-3 shows the change in strain from the time of setting of the concrete. As shown by comparing the two graphs, the embedment and bottom mount gauges have increased tensile strains in Figure 5-3 because the temperature when tensile strains that could cause stress started to accumulate was higher than when the concrete was placed, shown in Figure 5-2. Girder-mount gauges are unchanged because the girders were precast and therefore it was assumed that stresses could occur in the girders from the time of placement. It is apparent that when the concrete sets near its highest temperature it causes much higher tensile strains. This is discussed further in Section 5.3.2.

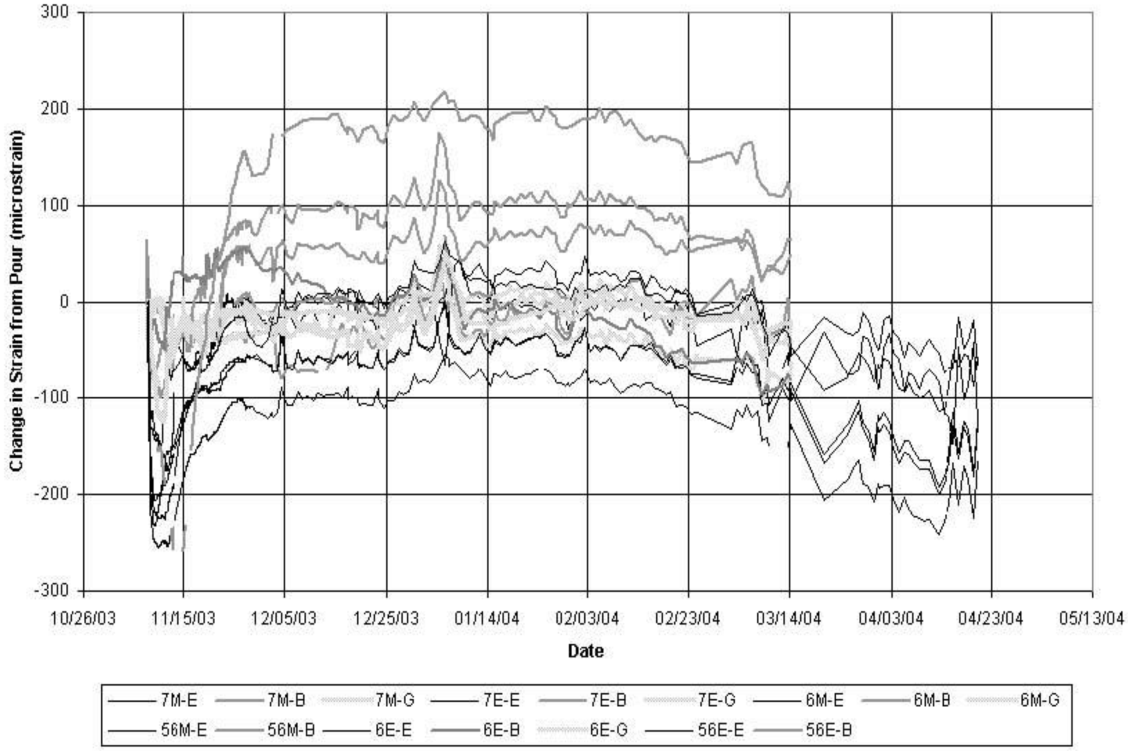

Figure 5-2: Gauge Strains Prior to Initialization.

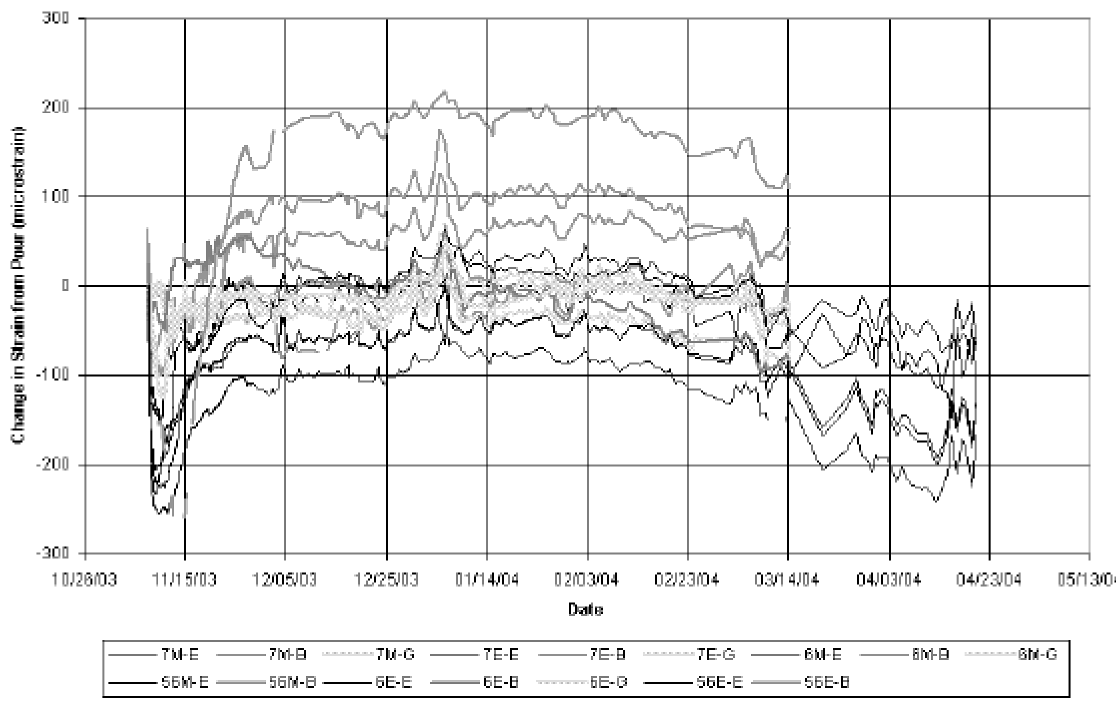

Figure 5-3: Gauge Strains after Initialization.

5.3.2 Calculated Strains

From the literature review it was determined that strains in the deck are caused from thermal changes, drying shrinkage, and creep. Using the data from the material tests in Section 3 and the recorded temperature data; the expected unrestrained strain change was calculated by summing the strains from drying shrinkage and thermal changes and subtracting the creep strains. Appendix B shows a comparison of the different causes of strain. As with the temperature data, the graphs were split into two time intervals for clarity.

The "Temp" strains in Appendix B graphs were calculated by taking the coefficient of thermal expansion of the concrete, determined in Section 3, and multiplying it by the temperature change from time of Initialization. Since the bottom mount and embedment gauges were zeroed when the deck was close to its peak hydration temperature, they reach much higher tensile strains from thermal changes than the girders. As the air temperature decreases, the tensile strain increases. The maximum tensile strain from temperature changes is shown in the even-numbered graphs in Appendix B near the time when the minimum air temperature was recorded (1/5/04). For all gauge locations, the thermal strains are the dominating strains.

The second strain-causing factor is drying shrinkage and is calculated by using Equation 3-5. This equation for drying shrinkage is for shrinkage of laboratory specimens. To determine the strains from drying shrinkage in the actual bridge deck, the equation was divided by a factor of 2.5 as discussed in Section 3.2.4. The age at which drying shrinkage was assumed to start depended on the gauge location on the deck. Embedment gauges were assumed to start shrinking when the deck insulating cover was removed on 11/17/03 and it was assumed that the bottom of the deck started shrinking when the forms were removed on 11/28/03. The girders were precast and were already around 60 days old when the deck was poured, so girder shrinkage was assumed to already be at an age of 60 days for use in the shrinkage equation; therefore 60 days was subtracted from the age variable in Equation 3-5. This causes the calculated shrinkage strains for the girders to increase much slower than the deck shrinkage strains.

With the temperature strains and shrinkage strains calculated, adding together the "free" thermal and shrinkage strains and subtracting the actual strains that occurred at each location provides a first estimate of the restrained strain in the deck. Multiplying the strain difference by the modulus of elasticity of the concrete at that time calculates the stresses at that location if no cracking occurred. The calculated stress is higher than the actual stresses in the deck because creep in the deck reduces the amount of restrained strain. To calculate the amount of creep that occurs in the deck and girders, the stress that was calculated was used with Equation 3-4. A program was created that first looked at whether the stress was compressive or tensile.

It then calculated the average of all the stresses up to the point when the location experienced a stress reversal. Using the age of the concrete and the average stress, the creep during that time interval could be estimated. When a stress reversal occurred, the program again calculated the average stress up to the next stress reversal or the end of the data. The creep strain was estimated by using the total age of the concrete with the derived equation for creep, multiplying it by the new average stress, and adding the creep strain that the concrete was at when the stress reversal occurred. In this way, a slightly overestimated creep strain in the deck over time was calculated. The overestimate is very minor, approximately 1 percent, however and should have minimal affect on the results.

5.3.3 Restrained Strains

Theoretically, the deck and girders should have strains equivalent to the summation of the thermal and shrinkage strains calculated in Section 5.3.2; however, the girders, abutments, parapet wall, and sidewalk all act to restrain the deck. If there were no restraint on the deck, it would shrink the amount calculated. Instead, the deck shrinks the amount recorded on the gauges. By subtracting the recorded strains from the theoretical strains, an estimate of the amount of restraint on the deck was determined. Since creep acts to alleviate stress in the concrete, the theoretical creep strains were then subtracted from the restraint estimate to give the total strain that produces stress in the deck. For the girders, the calculated strain was subtracted from the actual strain because shrinkage of the deck from thermal changes and drying shrinkage compress the girders. In this way, the girders act to cause tensile stresses in the deck and the deck acts to cause compressive stresses in the girders. Appendix C provides graphs that compare the actual strain to the summation of the calculated strain, minus the calculated creep strain, at each gauge configuration location. It should be noted these graphs may have gaps in the recorded strains that occurred when the data logger quit reading or the construction workers knocked the gauge off. The reattachment time was noted on the graph if it occurred. The actual strain graphs can also be used to determine when cracking occurred in the deck. This is discussed further in Section 5.4.

Table 5-3: Maximum Restraint at Each Gauge

Gauge ID	Maximum Tensile Restraint (microstrain)	Date Maximum Occurred
7M-B	341	01/05/2004
6E-B	333	01/05/2004
7M-E	313	01/05/2004
7E-B	294	01/06/2004
6M-B	286	01/06/2004
56M-E	282	01/06/2004
6E-E	279	01/05/2004
6M-E	272	01/06/2004
56E-E	267	01/05/2004
7E-E	250	01/06/2004
56M-B	211	01/06/2004
56E-B	165	11/14/2003
7E-G	-156	01/06/2004
7M-G	-162	11/23/2003
6E-G	-164	01/05/2004
6M-G	-177	01/06/2004

From the graphs in Appendix C and from Table 5-3, a general pattern can be noted that shows the largest restraint on the bottom of the deck, followed by the top, and finally the girders. As the distance from the girder increases, the restraint from that girder decreases, thus causing more restraint in the bottom of the deck than the top. The girders have less restraint because the stiffness of the deck relative to the girders is much less, so the girders provide comparatively more tensile restraint on the deck than the shrinkage of the deck produces compressive stress on the girders. At first glance, the gauges located between girders five and six seem to go against the general pattern; however, with the girders so far away from the gauges, drying shrinkage then becomes a larger factor in increasing the amount of restrained strain. Since the top of the deck dries faster than the bottom, more restraint is recorded on the top. Another general pattern that appears is that the deck near the exterior girders is restrained more than the rest of the deck. This is due to the extra restraint provided by the parapet wall and sidewalk in this location. A pattern between the middle span gauges and the end span gauges is not apparent. The gauges between Girders 5 and 6 and near Girder 7 show that the middle span has more restraint, but the end has more restraint on Girder 6. Varying degrees of drying shrinkage and thermal affects cause the restrained strain to differ in these locations.

5.4 Deck Stresses and Cracking

After the total restrained strain is calculated, the amount of stress in the deck can be estimated by multiplying the strain at a given time by the modulus of elasticity (E) of the concrete at that time. Since E changes with the age of the concrete, Equation 3-2 can be used to determine E at a given age for the deck concrete. The modulus of elasticity of the girders was assumed to conform to the relationship to compressive strength of Equation 2-1. At a compressive strength of 8000 psi, the girders have a modulus of elasticity of about 5,100,000 psi. The tensile capacity of the deck concrete at a given age was also calculated using the correlation between compressive strength and tensile strength, Equation 3-3, and the equation for compressive strength at a given age, Equation 3-1. Appendix D presents the potential stresses in the deck and girders, if no cracking occurred. For comparison purposes, the tensile capacity is also shown.

The actual stresses in the deck may differ dramatically from what is shown in Appendix D. The graphs in Appendix D graphs attempt to determine when the initial cracking started. Up to the first crack on the deck, the calculated stresses are fairly accurate. After a crack occurs, stress in the concrete is relieved at that location and transferred to the reinforcement. A reduction in stress of the surrounding concrete may also occur. The graphs from Appendix C and D can be compared to determine when cracking occurred. Theoretically the concrete should crack when tensile stresses exceed the tensile capacity of the concrete, which is shown in graphs in Appendix D. When a crack does occur, the tensile stresses and strains should drop suddenly.

The drop will be minor because any tension stresses in the concrete will be redistributed to the reinforcement. If the gauge is closer to the crack the drop will be dramatic. Another indicator of a crack occurring is a sudden change in the slope of the recorded strain line without a sudden change in the average daily deck temperatures.

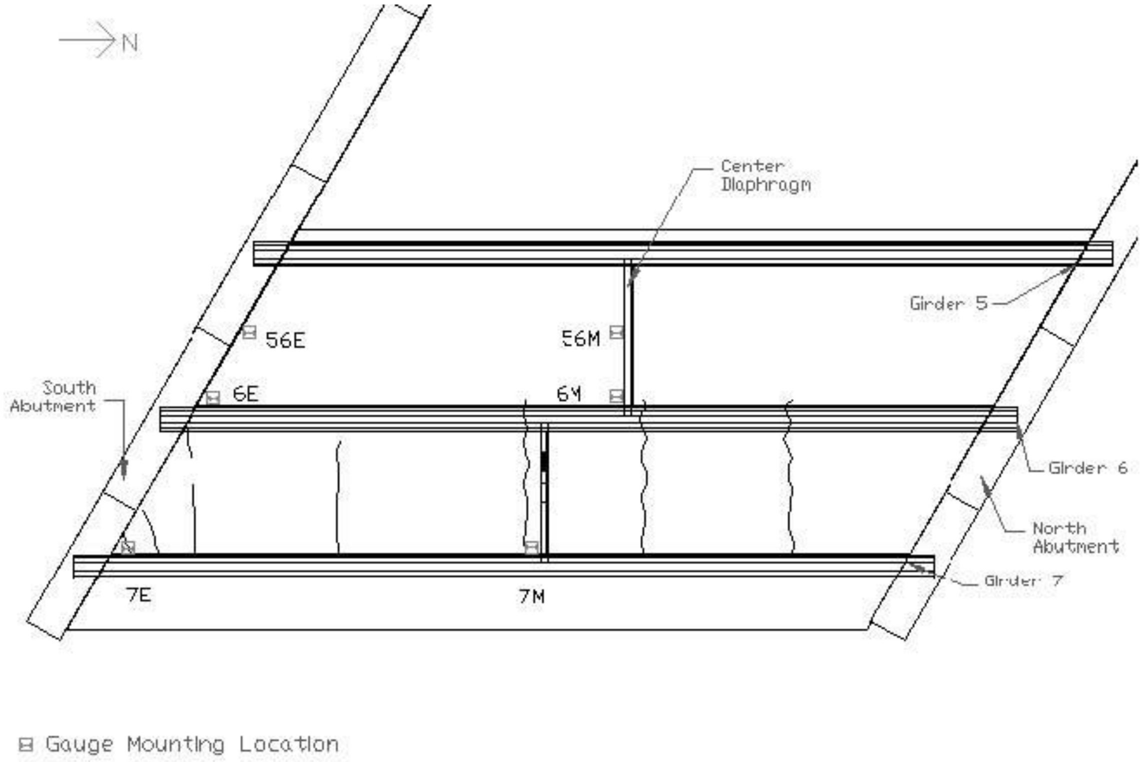

Illustration 5-2: Crack Locations Relative to Gauge Configuration Locations.

Illustration 5-2 shows were the cracks are located relative to the gauges. Note the concentration of cracking that occurs in the acute corner region. As shown, only gauges at locations 7M, 7E, and 6M are near a crack; however, stresses at other gauge locations may have been redistributed when cracking occurred. Visible cracking of the bridge deck was discovered on 3/22/04 after a rainstorm occurred. When the deck is dry or completely saturated the cracks are extremely difficult to detect. Only when the deck is partially moistened are the cracks easily detected; therefore, actual cracking of the deck could have occurred long before they were visibly noted. The majority of the remainder of this section will focus on cracking of the top of the deck by looking at the strain and stress data from the embedment gauges. Cracking of the bottom of the deck is expected and does not affect the overall durability of the deck.

Looking at the actual strain values is the easiest way to detect when cracking has occurred. In Figure 5-4 a possible time of cracking is shown on 11/20/04 at 4:00 p.m. Cracking can be assumed to occur because the actual strain line for gauge 7M-E has an abrupt change in direction. Although the direction change is minor, it occurs at the same time that the stress diagram, shown in Figure 5-5, exceeds the tensile capacity of the concrete and also has an abrupt drop in tensile stress. Following the initial crack is a series of additional drops in stress on 11/21/04 at 4:00 p.m. and 11/22/04 at 3:30 p.m. Both of these are indicated by extreme changes in the slope of the strain diagram.

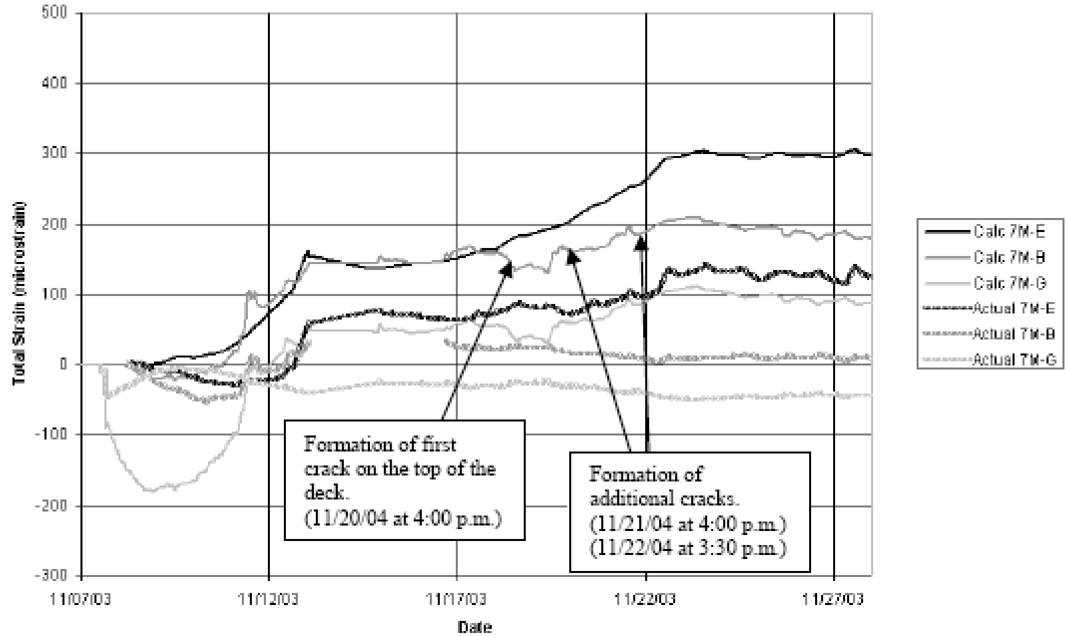

Figure 5-4: Calculated vs. Recorded Strain for Gauge Location 7M.

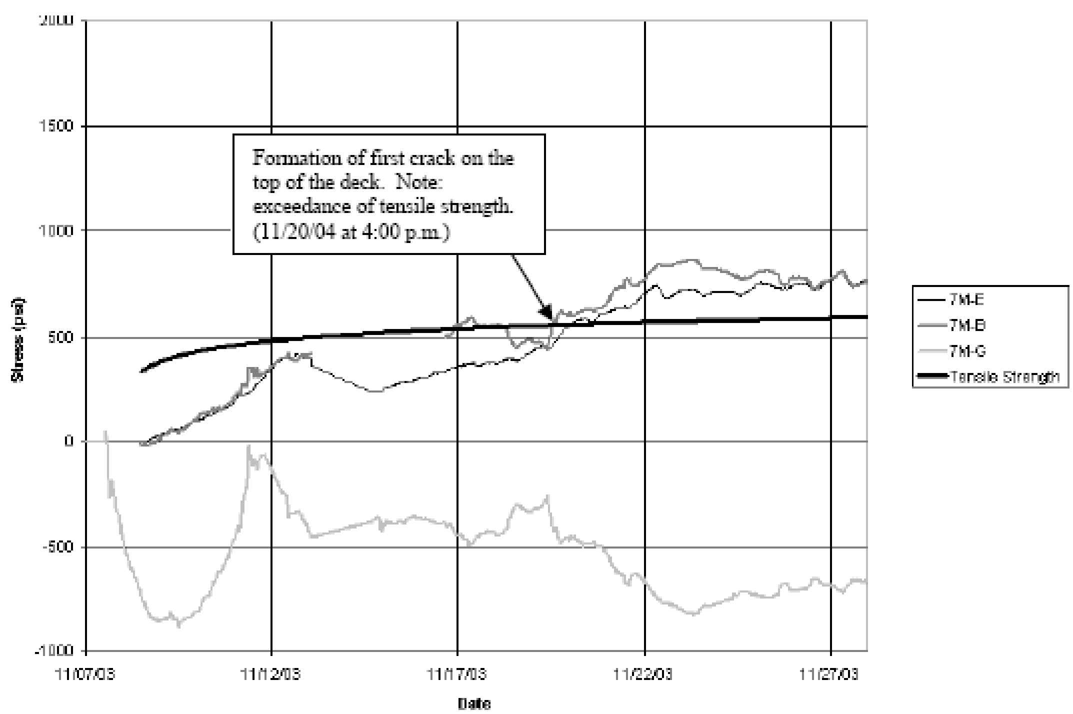

Figure 5-5: Potential Stresses in the Deck at Gauge Location 7M.

The greatest change occurs between the second and third stress relief cracking. At approximately the same time, gauge 7E-E also exceeded the concrete tensile capacity and showed abrupt changes in the slopes of both the stress and strain diagrams shown in Figures D-1 and C-1, respectively. This indicates that cracking was occurring near both areas and stresses were being redistributed at the same time. Although there are extreme changes in the slope of the strain diagrams at other times, they occurred when the stress was substantially below the tensile capacity of the concrete. Also, the additional slope changes occurred when there where abrupt changes in the deck temperatures, indicating stress changes from rapid thermal changes rather than a relief of stress.

The only other gauge that had any indication of cracking is 6M-E. On 11/22/03 at 11:30 a.m., the slope of the strain diagram suddenly began to increase, as shown in Figure C-5. The tensile strains increase with no indication of a change in thermal strains at this time. This change occurs between the time that the second and third stress relief indicators were occurring for the exterior girder gauges; therefore the slope change could be due to cracking at the edges of the deck or the drops in stress at the girder seven gauges could be due to cracking at gauge 6M. The stress had already exceeded the tensile capacity according to Figure D-5, so the calculated stresses shown in Appendix D were no longer valid after the cracking occurred at gauges 7M and 7E. Indications of cracking were not found at any of the other embedment gauge locations; however, the tensile capacity was exceeded at every location, according to the graphs in Appendix D. A redistribution of stresses must have occurred over the entire deck when the cracks shown in Illustration 5-2, and discussed in the previous paragraph, occurred.

6. –SUMMARY AND RECOMMENDATIONS

Section 6.1, summarizes the results of the US 95 bridge and the authors' opinions concerning the causes for the deck cracking. Section 6.2 presents the lessons learned from this study and provides recommendations for crack control on future projects.

6.1 Results of the US 95 Bridge

The main cause of tensile strains in the instrumented bridge deck appeared to be the thermal change of the deck as it cooled from the maximum hydration temperatures to ambient air temperature. Drying shrinkage, although minor for this concrete, added to the tensile strain. Restraint of the tensile strains in the deck was provided by the concrete girders, which is greater than the restraint provided by most girders because of the integral abutment design. The integral abutment design of this bridge caused the girders to restrain the deck more than if the girders were not integral with the abutments because of the fixed-fixed end condition that exists for lower level stresses such as those from thermal changes. Since the creep properties are low and the modulus of elasticity properties are relatively high for this concrete, the tensile strains that were restrained by the girders were able to cause tensile stresses that exceeded the tensile rupture strength of the concrete in a relatively short amount of time. Cracking of the Stage deck 1 had occurred within 13-days after placement of the concrete. Due to higher creep properties, the Stage 2 deck performed slightly better. Cracking of the Stage 2 deck had occurred within one month after placement of the concrete.

The high hydration temperature for the Stage 1 deck was due to the mix design, which had high cement content and no fly ash to reduce the rate of hydration; therefore, the mix was prone to rapid hydration. In addition, the deck insulation and heating of the underside of the deck, due to cold weather concrete placement, retained the heat produced by hydration of the concrete. The rate of heating under the deck should have been reduced as the temperatures increased from hydration of the cement in the deck to reduce the maximum hydration temperature. The initial heating of the girders before placement was also beneficial in reducing tensile restraint on the deck from the girders; however, the recommendation in Section 1.3.1 that limits the girder/deck temperature differential to less than 22° F for the first 24 hours after placement is not sufficient for integral abutment bridges. For integral abutment bridges, the maximum hydration temperature has more of an effect on cracking than the temperature differential between the deck and girders.

6.1.1 Cracking at Acute Corners

Additional cracking near the South East corner of the Stage 1 deck was from traffic loads producing uplift at the acute corner, due to the relatively large skew of the bridge. This cracking is shown in Illustration 6-1. The parapet wall, abutments, girders, and sidewalk all served to restrain the uplift and cause cracking; therefore, the recommendation limiting skew to less than 30° to minimize the risk of cracking, given in Section 1.3.1, will have to be reduced for bridges designed such that the parapet wall and sidewalk are integral with the bridge deck. Additional research will need to be performed to determine the appropriate skew limit.

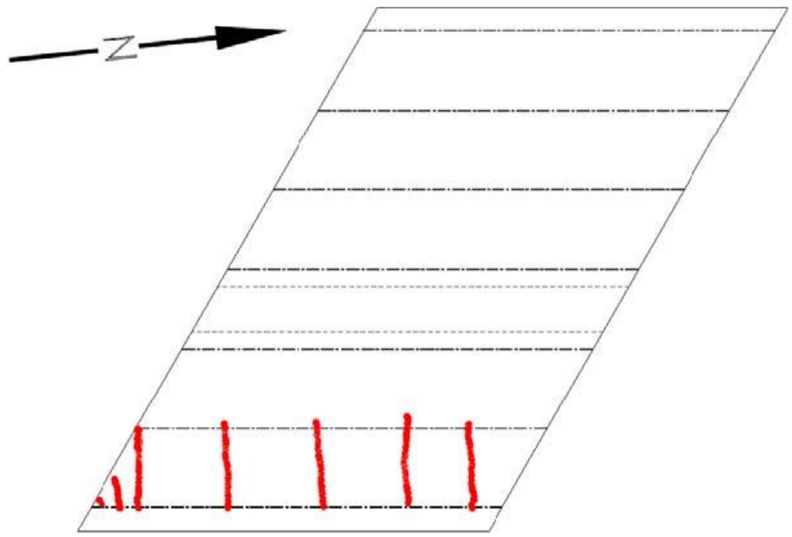

Illustration 6-1: Stage 1 Deck Cracking

After the completion of the Stage 2 deck, similar cracking occurred at the acute corner on the North West corner of the bridge deck, as shown in Illustration 6-2. The details of the cracking in the acute corner of the Stage 2 deck are shown in Illustration 6-3 and Illustration 6-4.

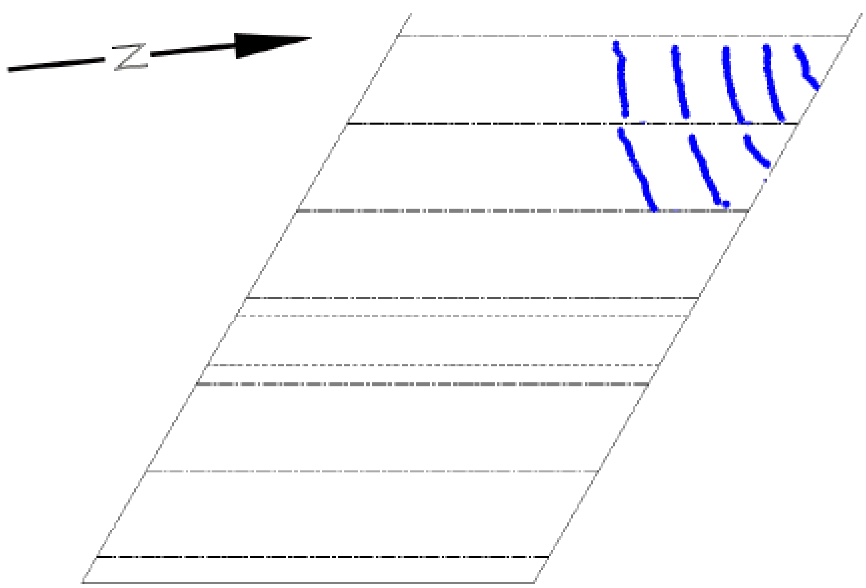

Illustration 6-2: Stage 2 Deck Cracking

Illustration 6-3: Stage 2 Deck Cracking Between Girders 1 and 2.

Illustration 6-4: Stage 2 Deck Cracking Between Girders 2 and 3.

The AASHTO LRFD Bridge Design Specifications contains the following recommendation for skewed bridge decks:

> If the skew exceeds 25°, the specified reinforcement in both directions shall be doubled in the end zones of the deck. Each end zone shall be taken as a longitudinal distance equal to the effective length of the slab specified in Article 9.7.2.3[(19)].

The 2001 revision of the ITD Bridge Design Manual defines the end zone for skewed bridges as shown in Illustration 6-5[20].

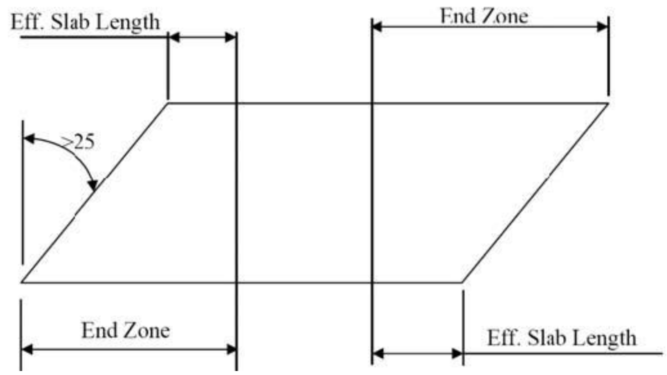

Illustration 6-5: ITD Definition of Skewed End Zones.

The US 95 bridge considered in this report has a skew of 30°. Due to the extreme width of the bridge compared to its length, the transverse reinforcing was doubled along the entire length of the bridge and the longitudinal steel was doubled in the end zone[17]. This additional longitudinal end zone reinforcement can be seen in the upper portion of Illustration 6-6. This illustration also shows that outside of the end zones, the longitudinal reinforcement spacing is twice the transverse spacing. Due to the skew and extreme bridge width, the end zone for the additional longitudinal steel was not squared off as depicted in the ITD bridge design manual (Illustration 6-5), but was provided in a region parallel to the ends of the bridge, as shown on Illustration 6-7.

Illustration 6-6: US 95 Bridge-Deck Reinforcement.

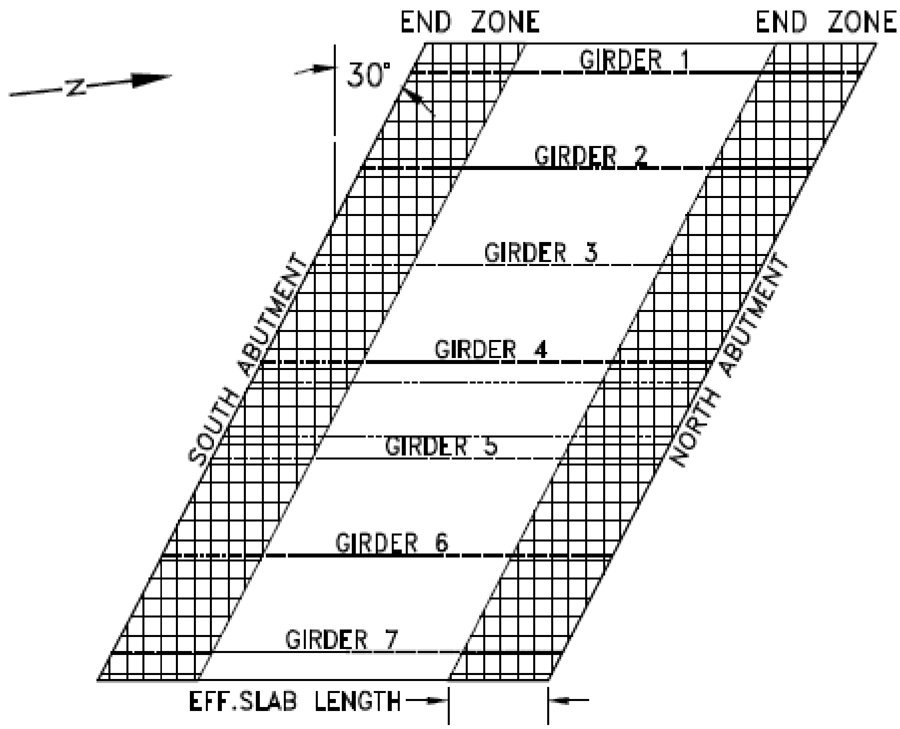

Illustration 6-7: US 95 Bridge - Location of Additional End Zone Reinforcement.

As shown in Illustration 6-1, Illustration 6-2, Illustration 6-3, and Illustration 6-4, cracking extended significantly past the location of the additional longitudinal end zone reinforcement.

6.1.2 Cracking at Pour Closure Section

Transverse cracks also occurred at regular intervals throughout the entire length of the closure pour section, as shown in Illustration 6-8 and Illustration 6-9. The closure pour section is continuously restrained along its edges by the hardened Stage 1 and Stage 2 deck concrete. Using development length as an analogy, the first cracks occurred when the strains induced by shrinkage exceed the tensile strength of the concrete. As shown in Illustration 6-10, the cracking will occur at evenly spaced intervals along the entire deck length[(21)].

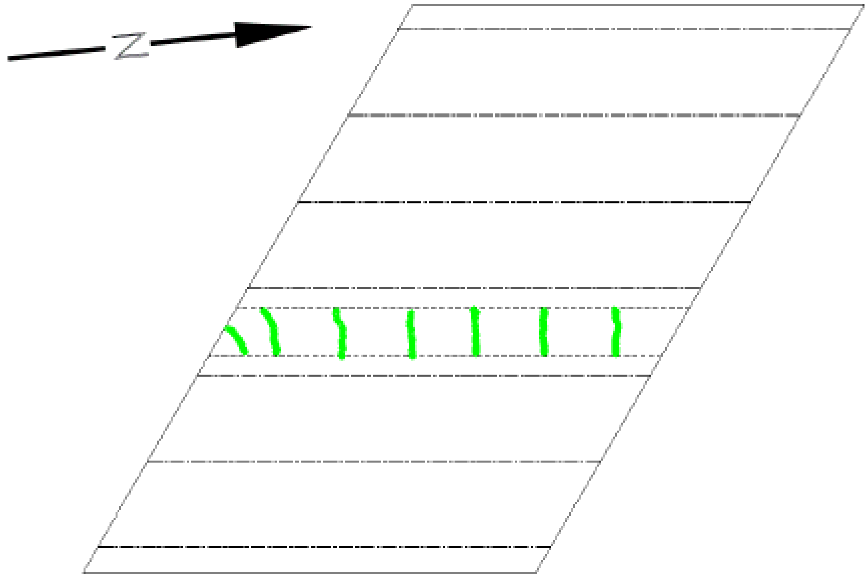

Illustration 6-8: Closure Pour Deck Cracking.

Illustration 6-9: Closure Pour Deck Cracking.

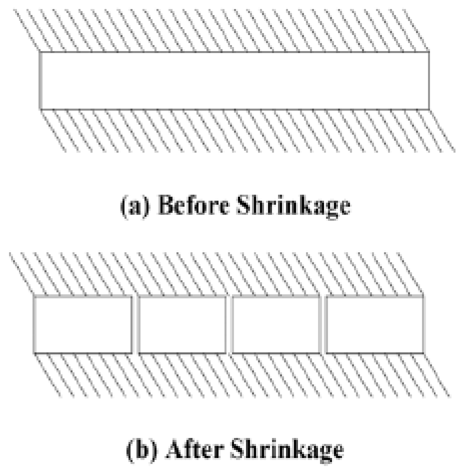

Illustration 6-10: Concrete Slab Restrained Along its Full Length.

6.1.3 Long-Term Considerations

At the time the report was written many of the cracks were sufficiently wide to allow the infiltration of deicing chemicals and moisture into the interior of the bridge deck, as shown in Illustration 6-3 and Illustration 6-4. Ultimately, traffic load cycles will cause the crack widths and sizes to further increase in size. As the cracks grow, the high permeability of the concrete will allow the chloride ions to reach the rebar much faster than other concretes, but the epoxy coating on the top reinforcement should decrease the amount of deterioration. The high scaling resistance, freeze-thaw durability, and abrasion resistance will also aid in reducing the amount of deterioration of the concrete as the deck ages. Special attention should be given to the cracks caused by the skew of the bridge, as these are the most likely to grow from traffic loading and are currently the widest cracks on the deck.

6.2 Recommendations

This section presents recommendations based upon the literature review and the observations of the US 95 bridge.

6.2.1 Comparision between US 95 Bridge and Recommendations from Literature Review

A comparison between the US 95 bridge deck concrete to the recommendations found in the literature review is shown in Table 6-1. As shown in this table, the cementitious material contained in the Stage 2 deck concrete was comparable to the recommended values. As was expected, the conventional mix design exceeded these design recommendations, which were reflected in the increase the shrinkage potential of the conventional concrete. The air content, skew, and use of water during curing were all within the recommendation ranges. The recommendations concerning reinforcing alignment was made to avoid planes of weakness in the uncured concrete. This recommendation warrants further consideration and possible adoption by ITD. The recommendation concerning the placement of the top layer of transverse reinforcement below the top longitudinal bars is more controversial. This recommendation, by Kraus and Rogalla[9] and by Shing and Abu-Hejleh[12], is not generally accepted by others, since following this recommendation would prohibit the use of the AASHTO Empirical Method for deck design. AASHTO LRFD Section 9.7.2.5 requires that the outermost layers be placed in the direction of the effective length.[19]

Table 6-1: Comparison Between US 95 Bridge and Recommended Practice

Item	Practice Used in Stage 1/Stage 2	Recommended Practice
Cementitious Materials	Stage 1 Deck Concrete 693 lb/yd^3 (Idaho Class 40A) Stage 2 Deck Concrete 583 lb/yd^3 (with 20% fly ash)	• 560 lb/yd^3 (Idaho Class 40AF) • Require fly ash (20% by weight of cementitious materials)
Air Content	Stage 1 Deck Concrete 5.3% by volume Stage 2 Deck Concrete 5.8% by volume	• 6% or higher by volume
Concrete Strength	**Stage 1 Deck Concrete** • 3100 psi over design strength at 28-days • 4200 psi over design strength at 90-days **Stage 2 Deck Concrete** • 3100 psi over design strength at 28-days. •2300 psi over design strength at 90-days.	• Reduce 1200 psi over strength requirement • Allow 56-days to reach design strength • Limit maximum compressive strength at 90-days
Reinforcement Alignment	• Top and bottom bars aligned vertically in transverse and longitudinal directions	• Stagger vertical alignment of top and bottom reinforcement
Top Transverse Bar Location	• Top transverse bars above the top longitudinal bars	• Place top transverse bars below top longitudinal bars [9,12] See Note 1
Skew	• 30°	• Reduce skew limit to less than 30° • If parapet wall and sidewalk are integral with the deck, limit skew to significantly less than 30°
Curing	• Geofabric and vinyl insulating cover • AASHTO "Water Method" for a minimum of 10 days	• AASHTO "Water Method" for a minimum of 7-days
Cold Weather Concreting	• Top of Stage 1 deck under insulation reached 100°F to 107°F. • Temperatures near the bottom of the Stage 1deck in the enclosure reached 80° F to 85° F.	• Maintain enclosure and insulated concrete cover temperatures between 55° F and 75° F.

Notes 1) The recommendation to place the top layer transverse bars below the top longitudinal bars, would allow the longitudinal bars to be more effective in reducing the transverse cracks. However, the use of this recommendation is controversial since it would prohibit the use of the AASHTO Empirical Method. AASHTO LRFD Section 9.7.2.5 requires that the outermost layers be placed in the direction of the effective length.[19]

6.2.2 **Bridge Design Recommendations**

With the exception of the item concerning the placement of longitudinal steel and the items concerning deck restraint, the bridge design recommendations listed in Table 1-3 are appropriate means for reducing transverse bridge deck cracking. Cover, maximizing deck thickness, maximizing girder spacing, and minimizing deck reinforcement spacing all appear to contribute toward improved deck performance. The cracking in the closure pour region of the deck can only be addressed by using a concrete with a low shrinkage potential and a high creep coefficient.

6.2.3 Material Characteristics Recommendations

Based upon the literature review and observations made of the US 95 bridge deck, it would appear that the shrinkage potential and creep characteristics of the deck concrete are the two major material characteristics that contribute towards the performance of the deck. The concrete mix design recommendations listed in Table 1-4 are appropriate means for reducing transverse bridge deck cracking. The use of low heat of hydration concrete, reduced water/cement ratios, fly ash, retarders, and Shrinkage Reducing Admixtures (SRA) should all be considered in order to reduce shrinkage. The modulus of elasticity and the compressive strength should both be as low as possible, consistent with the strength and deflection requirements of the deck. The rate of strength increase and the maximum strength should be limited.

6.2.4 Construction Procedure Recommendations

Based upon the literature review and observations made of the US 95 bridge deck, it would appear that the proper finishing and curing of the deck concrete are two major construction factors that contribute towards the performance of the deck. The concrete mix design recommendations listed in Table 1-5 are appropriate means for reducing transverse bridge deck cracking.

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APPENDIX A. DECK AND AIR TEMPERATURE GRAPHS

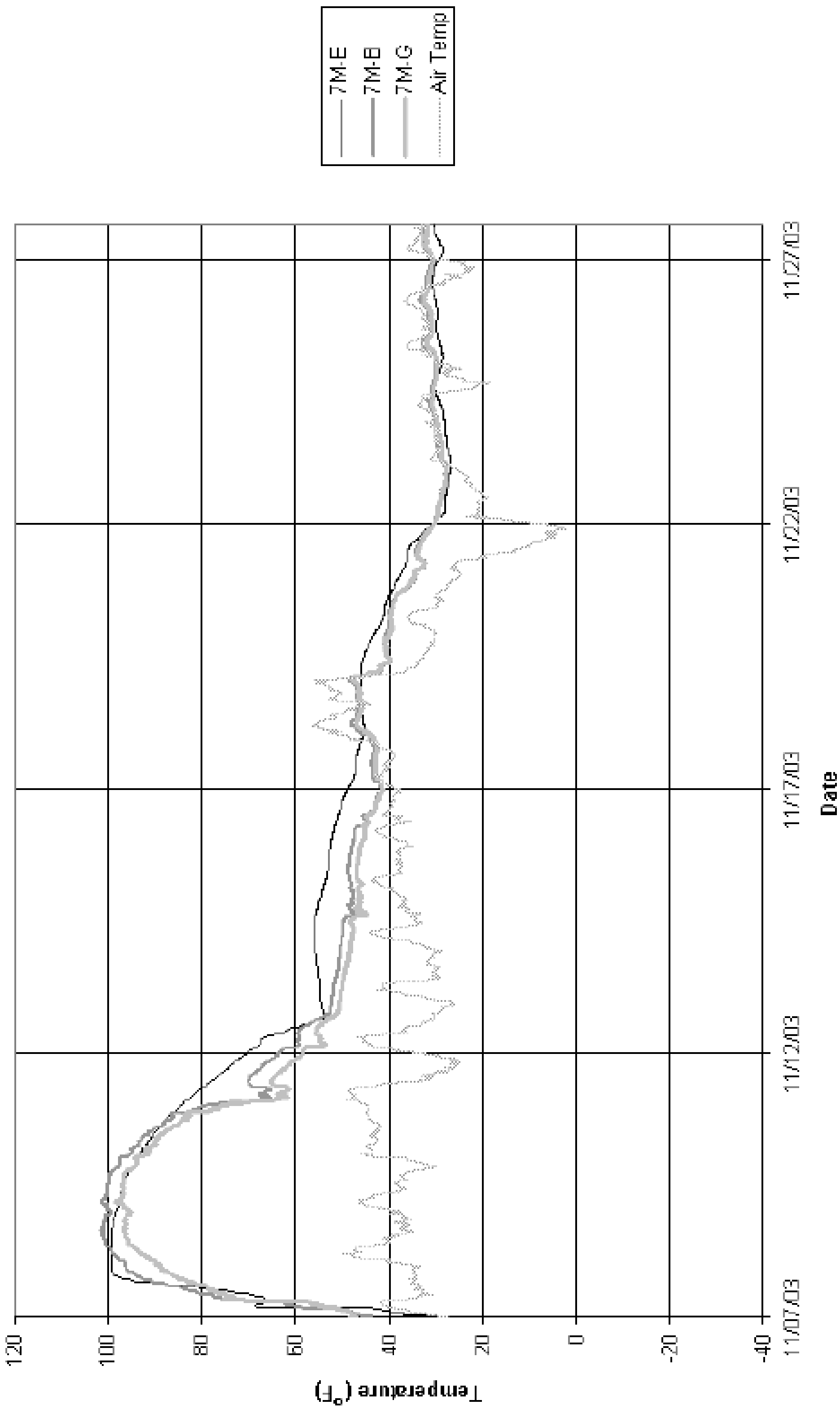

Figure A-1: Deck Temperatures at Gauge Location 7M (Time Interval 1).

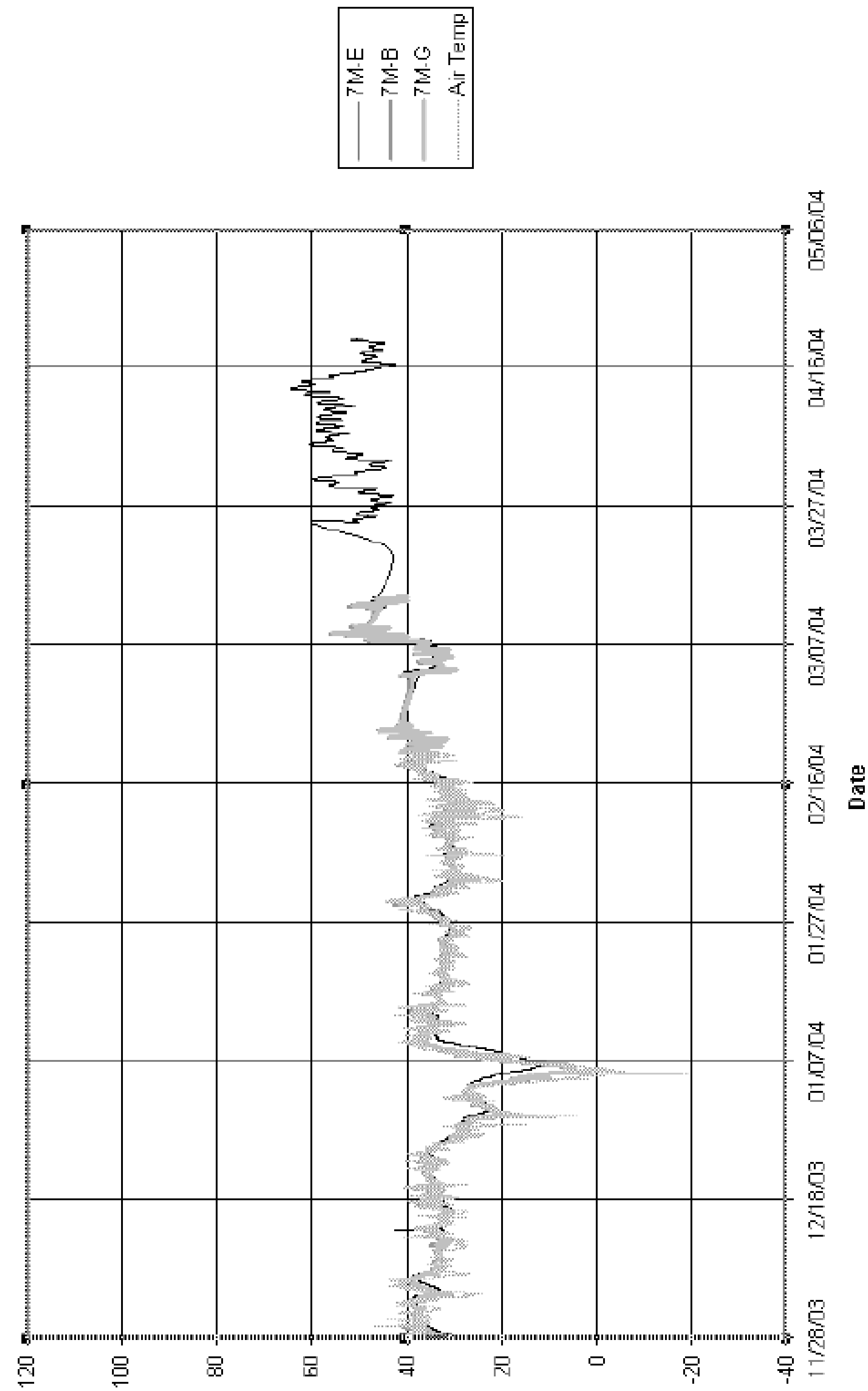

Figure A-2: Deck Temperatures at Gauge Location 7M (Time Interval 2).

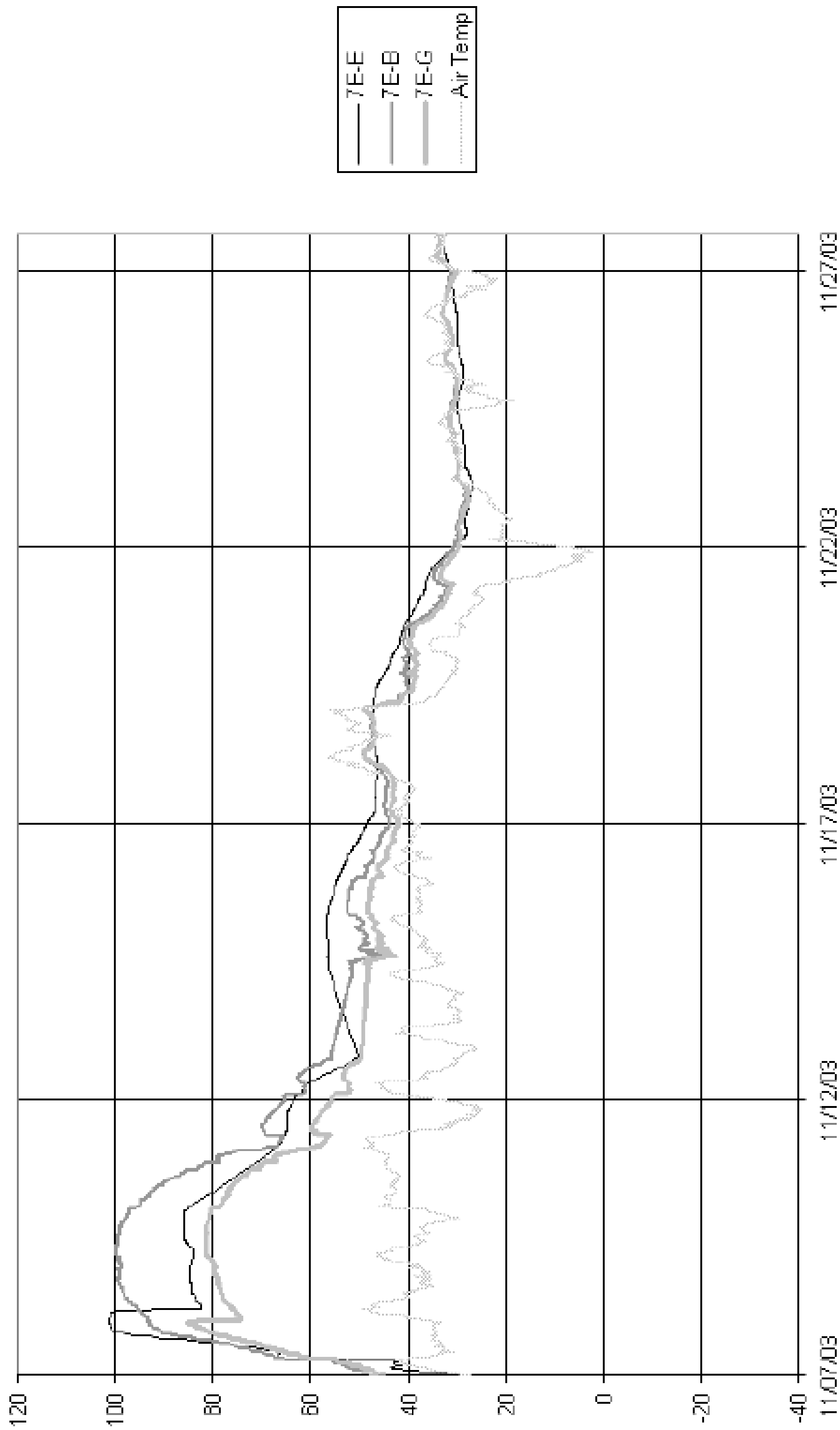

Figure A-3: Deck Temperatures at Gauge Location 7E (Time Interval 1).

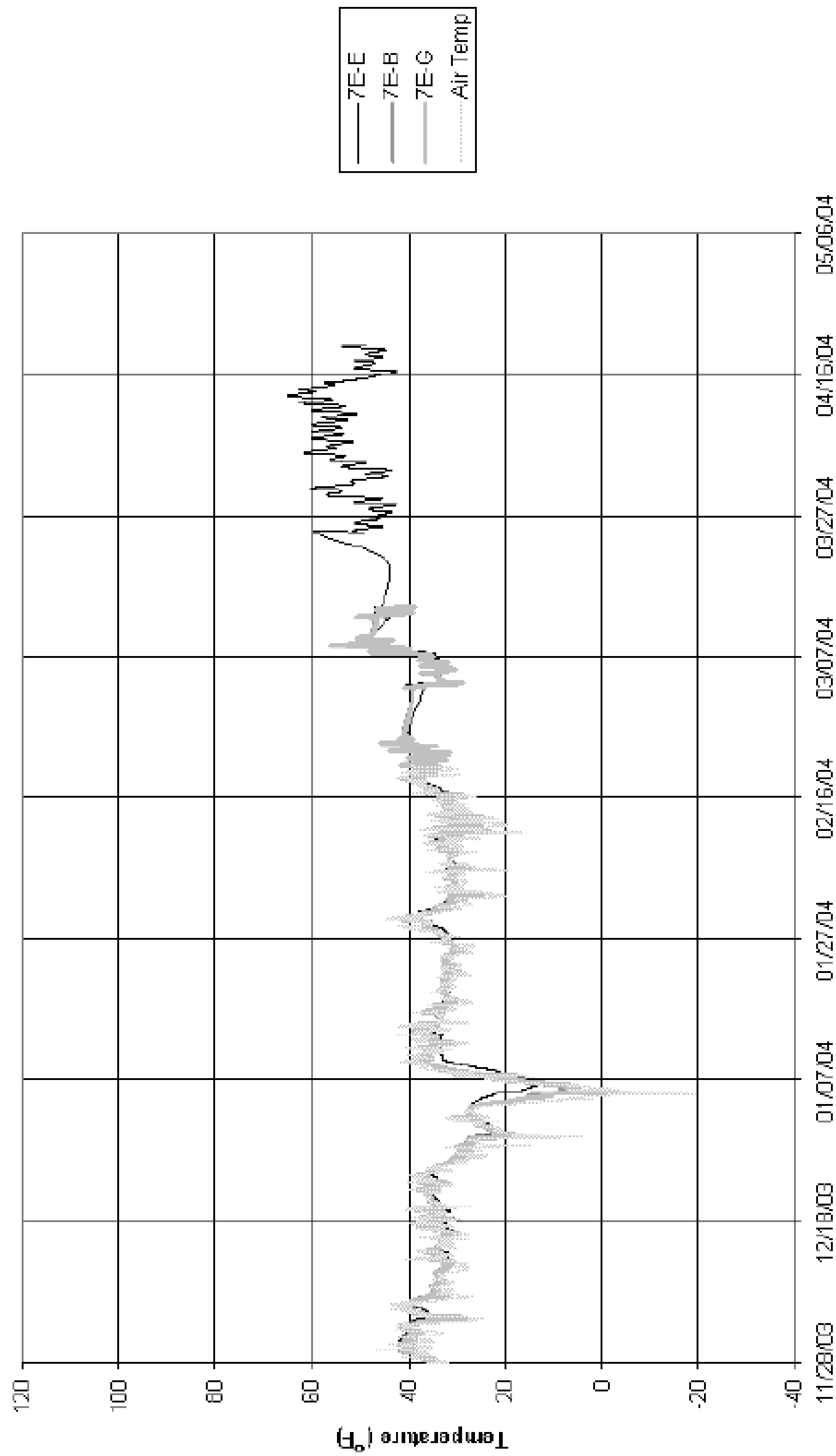

Figure A-4: Deck Temperatures at Gauge Location 7E (Time Interval 2).

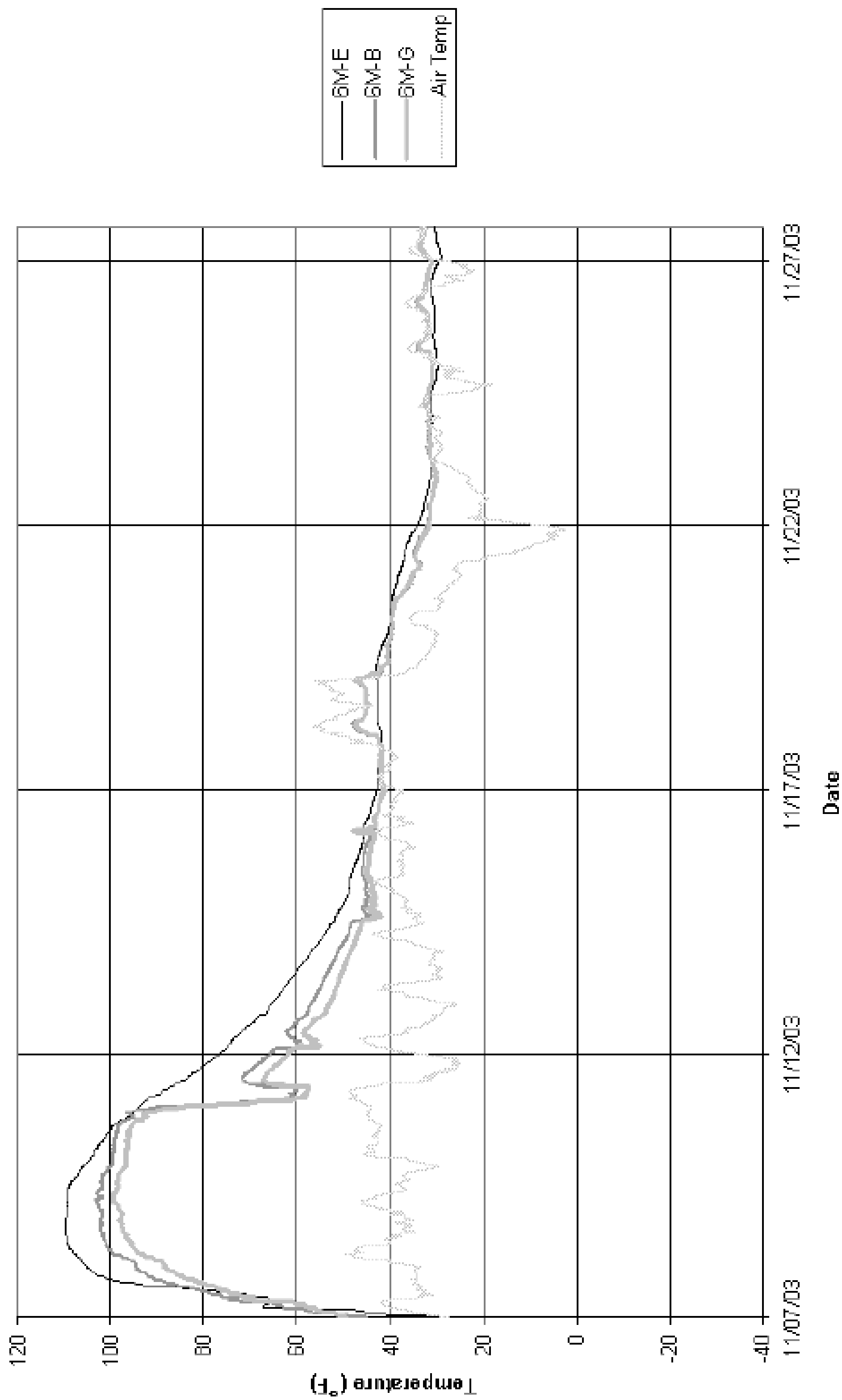

Figure A-5: Deck Temperatures at Gauge Location 6M (Time Interval 1).

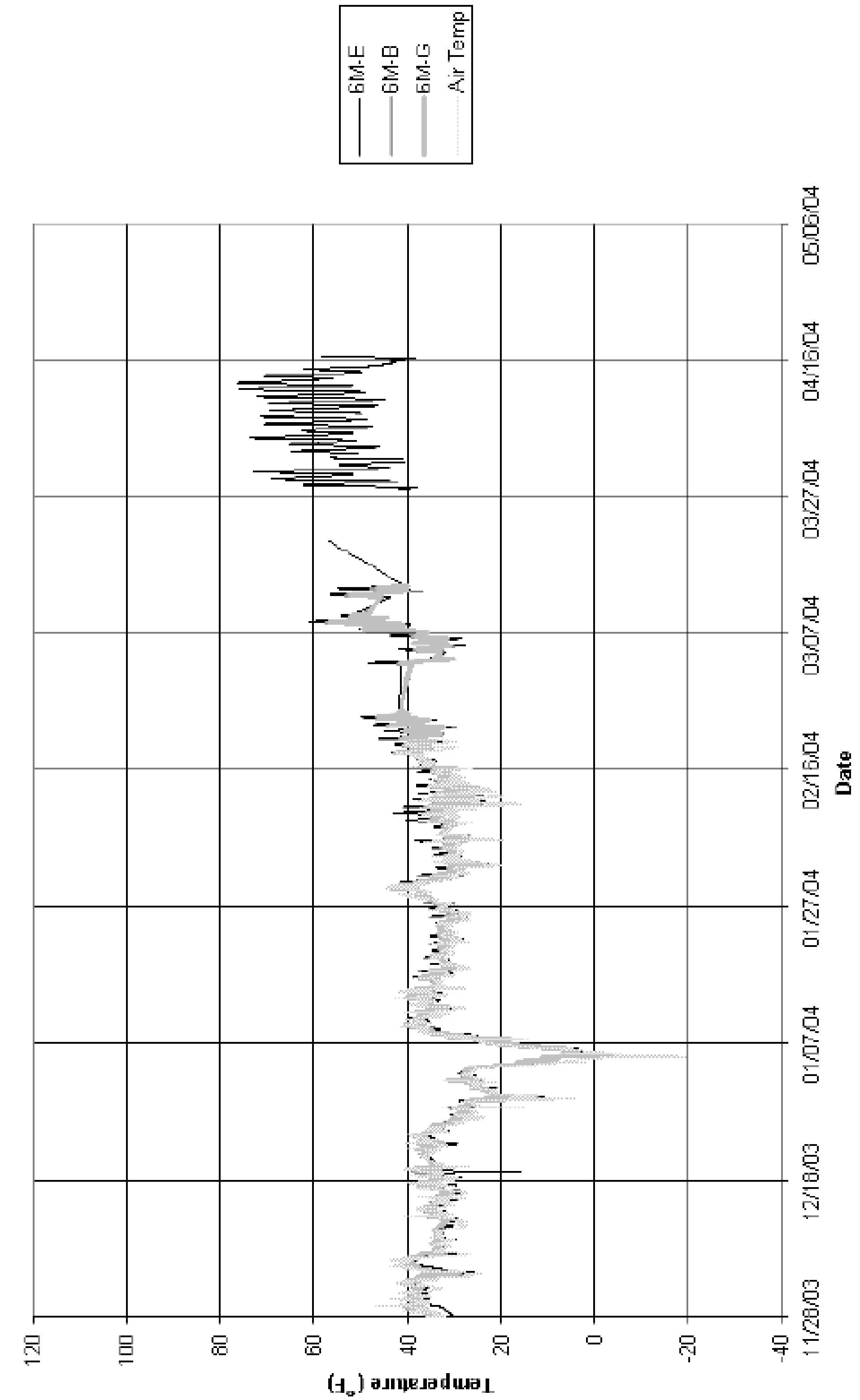

Figure A-6: Deck Temperatures at Gauge Location 6M (Time Interval 2).

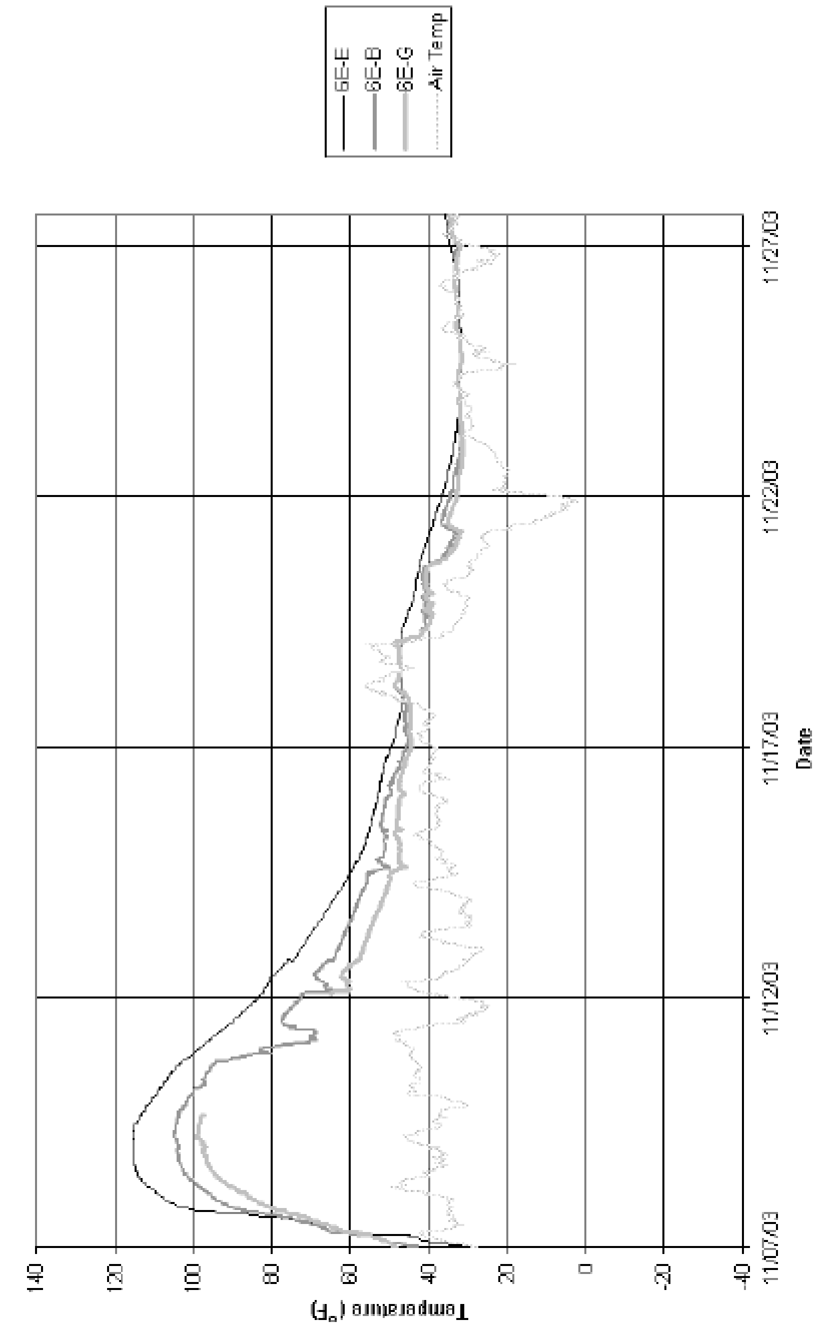

Figure A-7: Deck Temperatures at Gauge Location 6E (Time Interval 1).

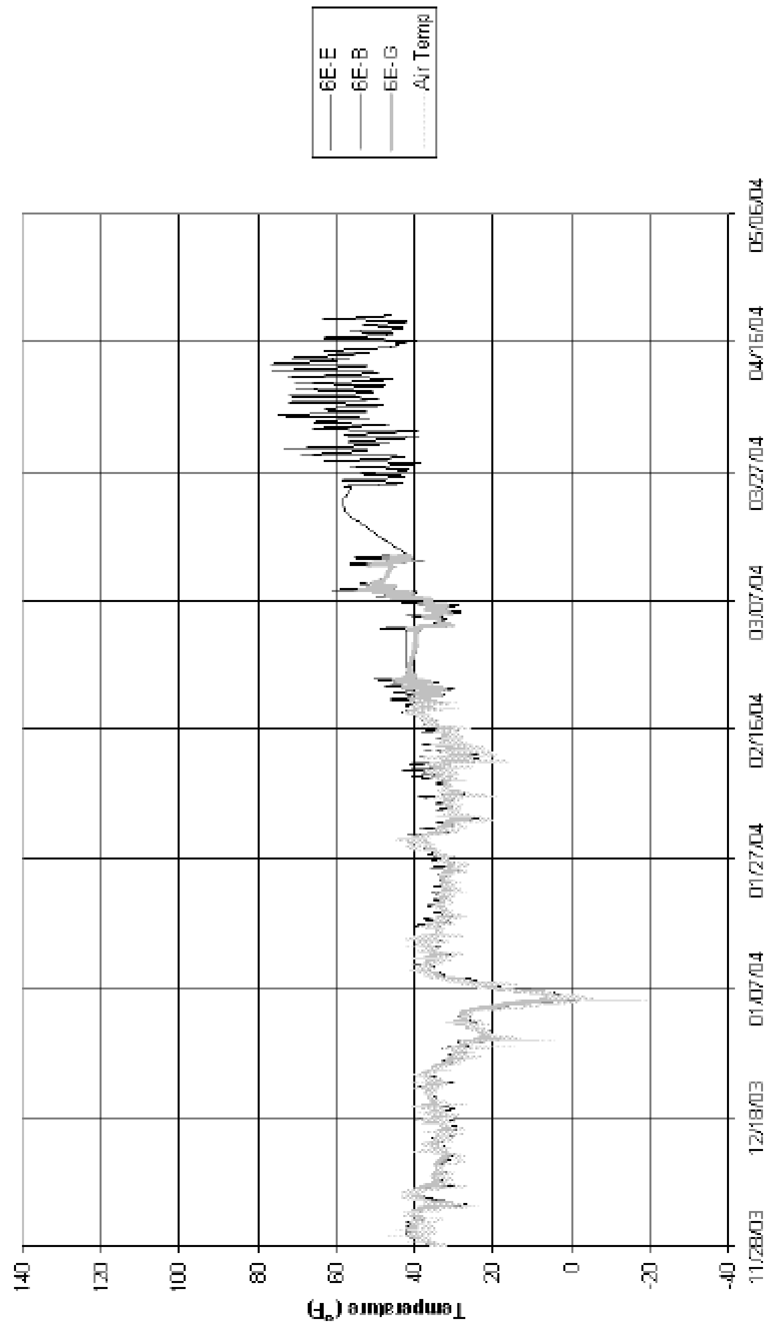

Figure A-8: Deck Temperatures at Gauge Location 6E (Time Interval 2).

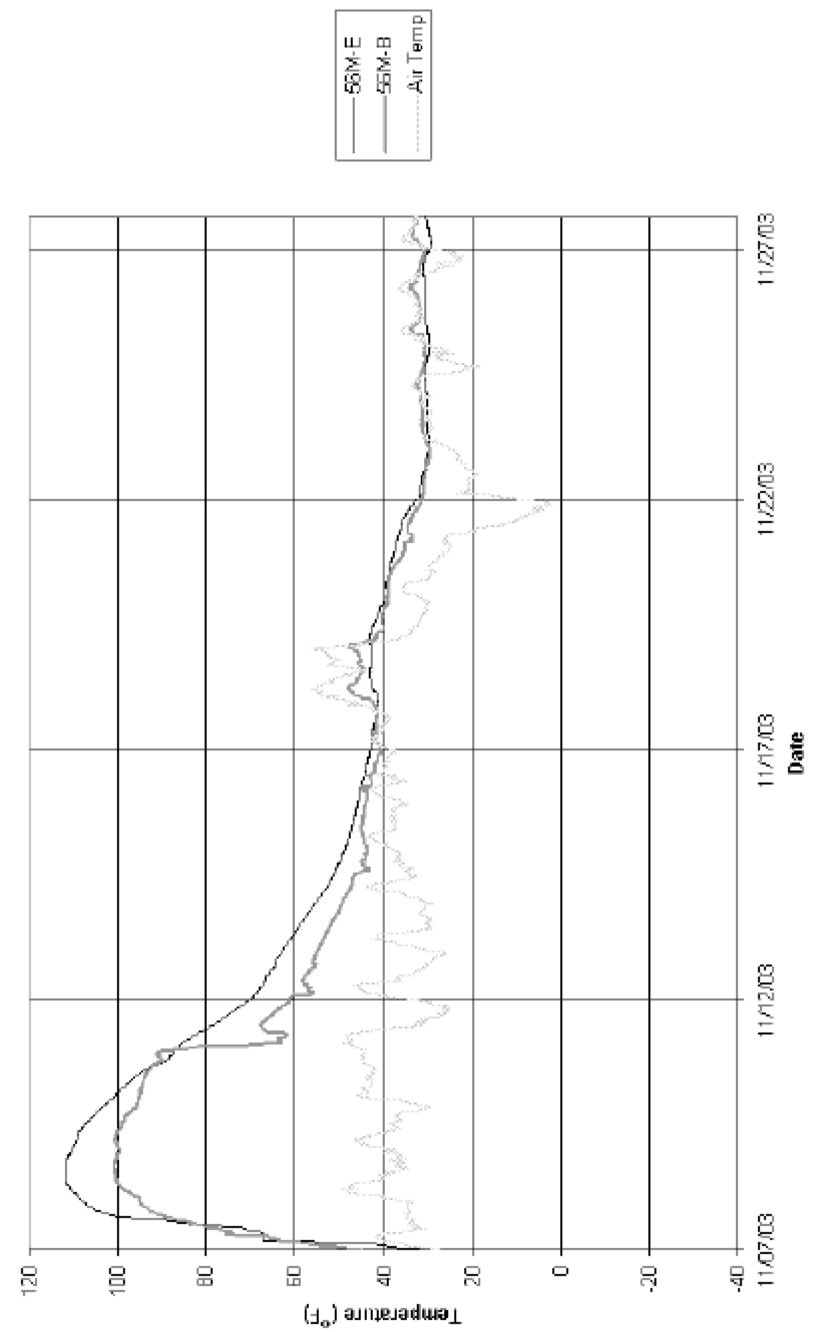

Figure A-9: Deck Temperatures at Gauge Location 56M (Time Interval 1).

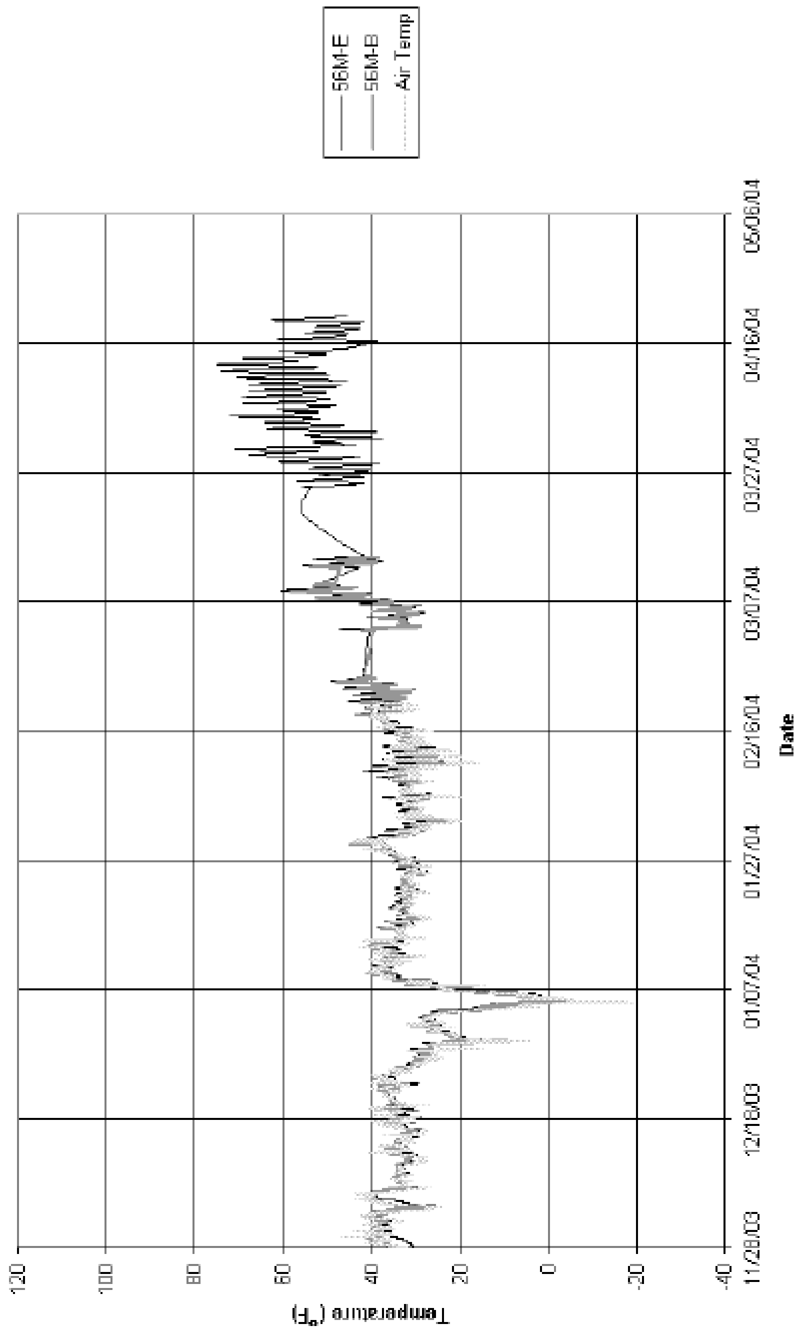

Figure A-10: Deck Temperatures at Gauge Location 56M (Time Interval 2).

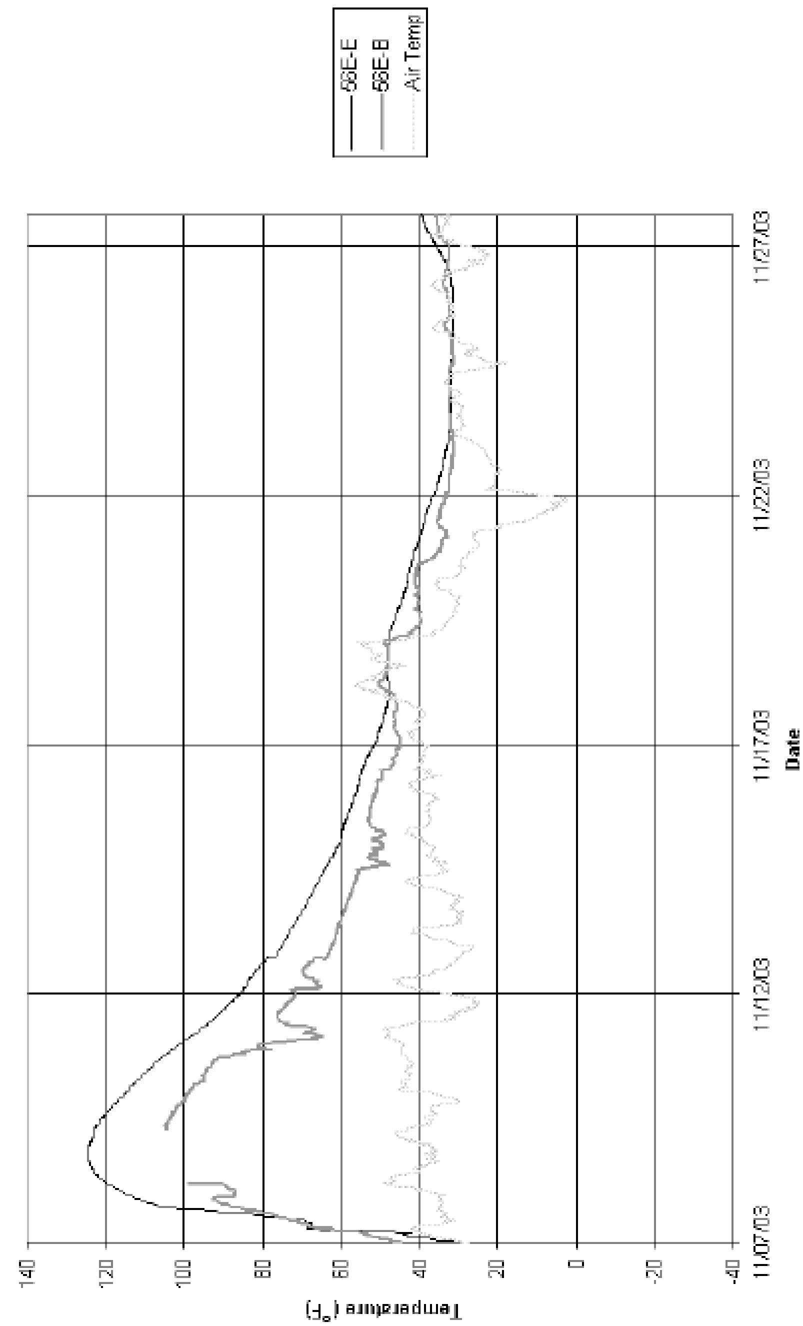

Figure A-11: Deck Temperatures at Gauge Location 56E (Time Interval 1).

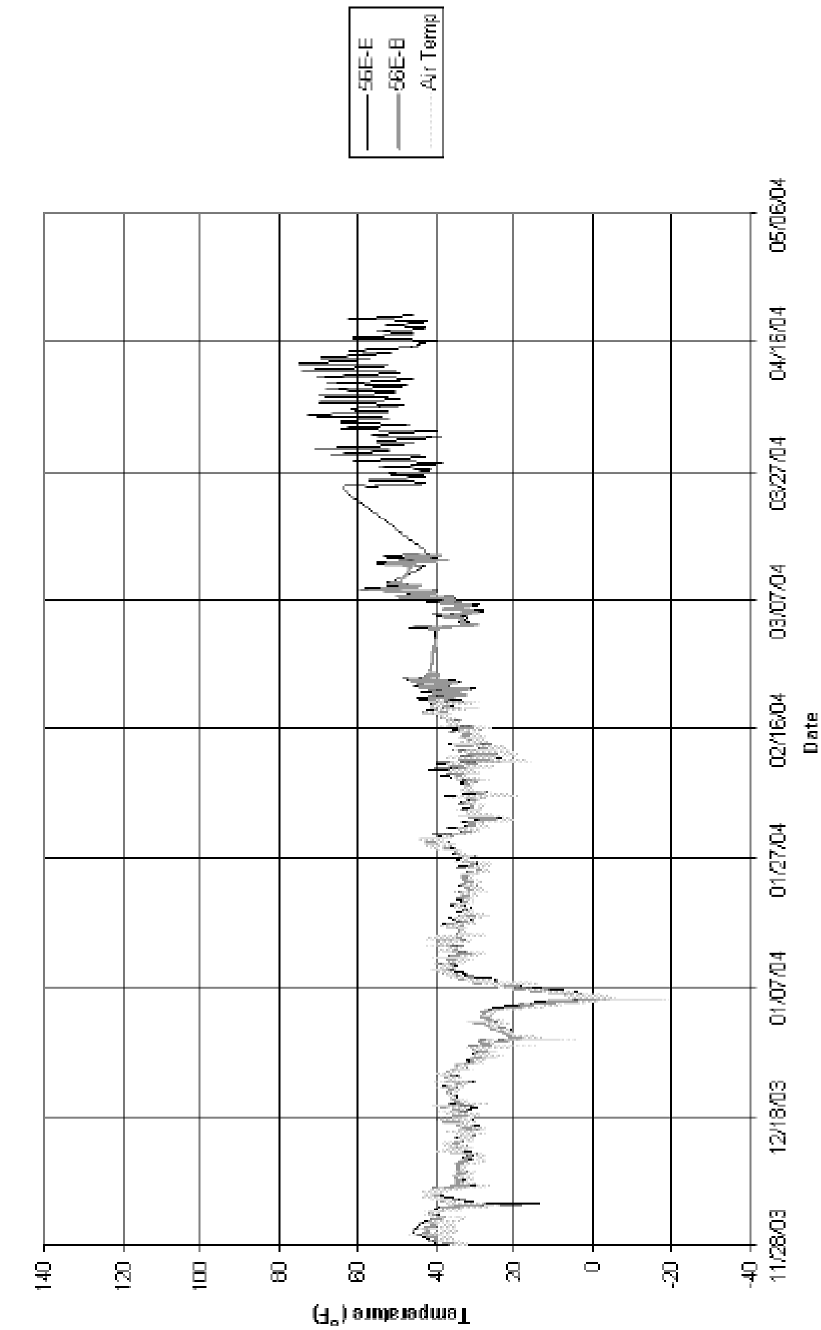

Figure A-12: Deck Temperatures at Gauge Location 56E (Time Interval 2).

APPENDIX B. CALCULATED STRAIN COMPARISON GRAPHS

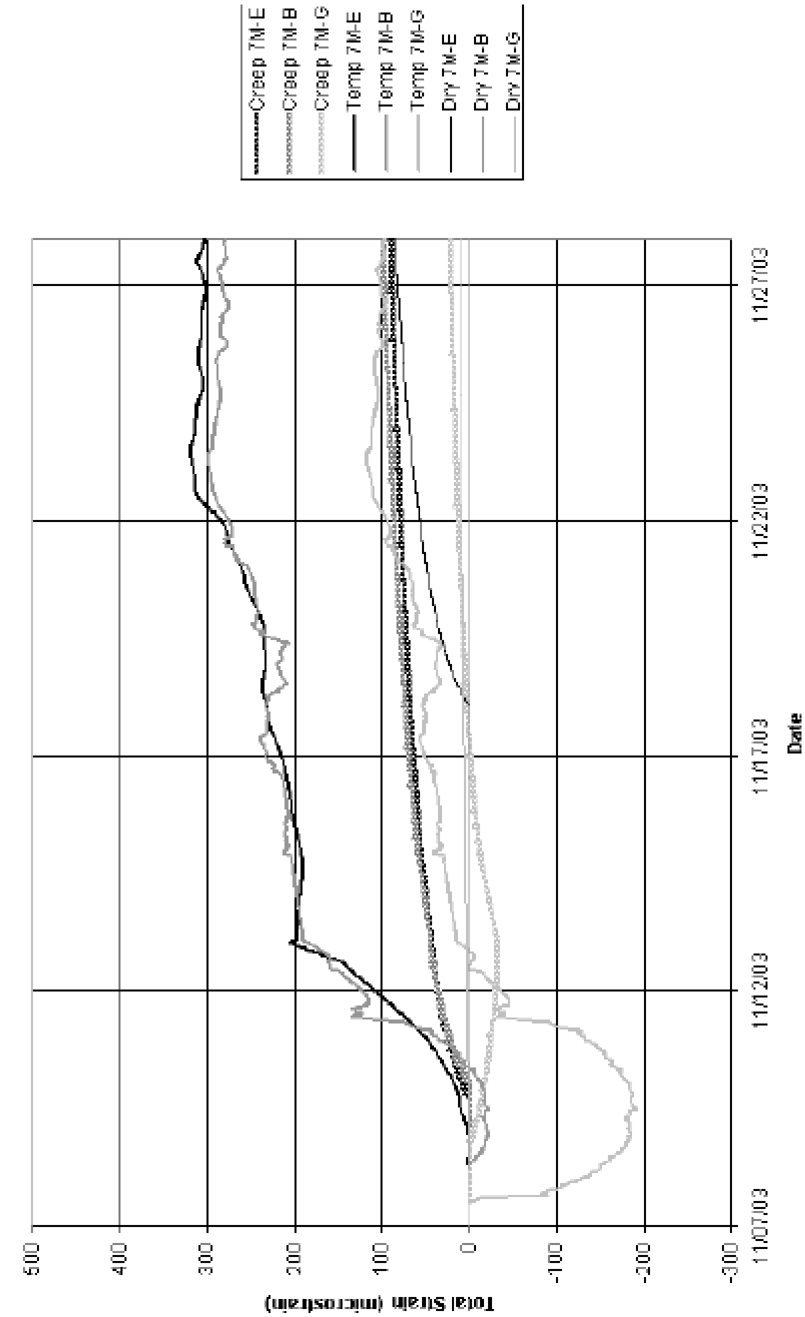

Figure B-1: Comparison of Calculated Strains for Gauge Location 7M (Interval 1).

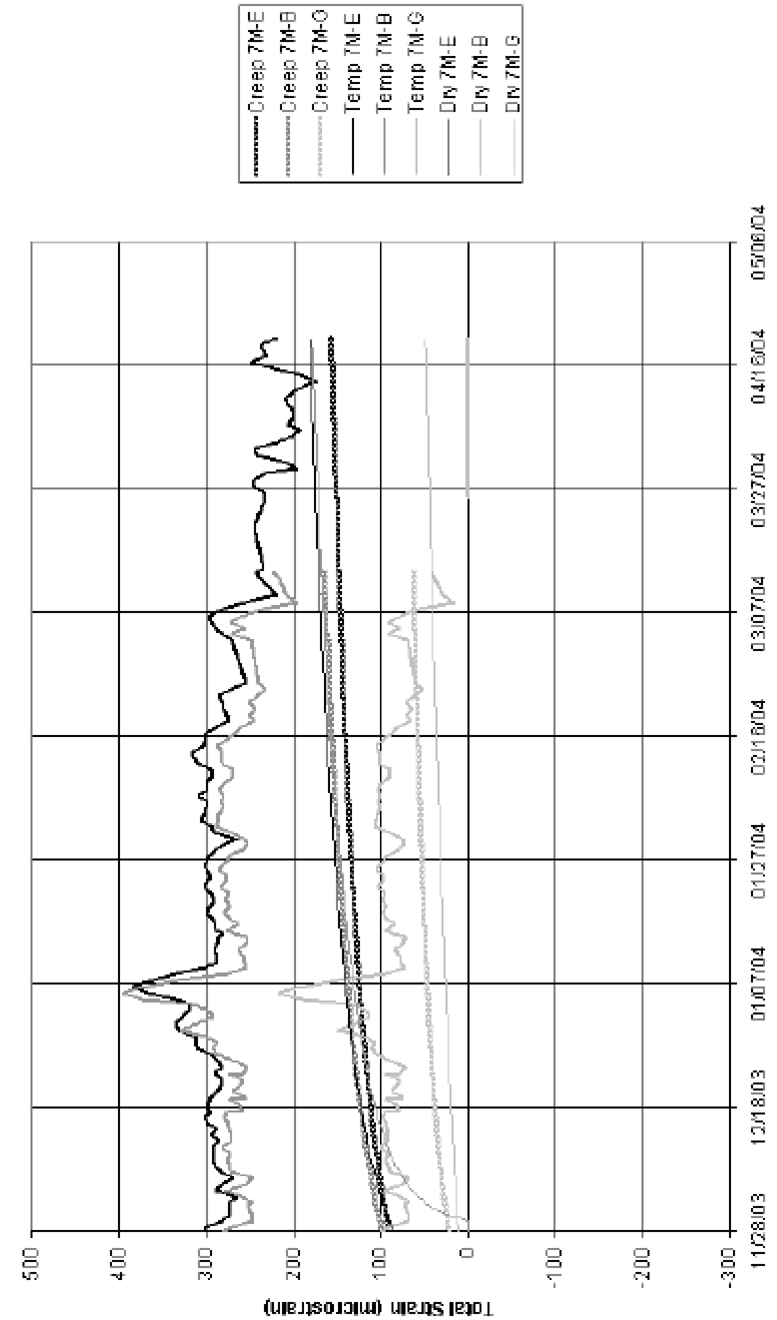

Figure B-2: Comparison of Calculated Strains for Gauge Location 7M (Interval 2).

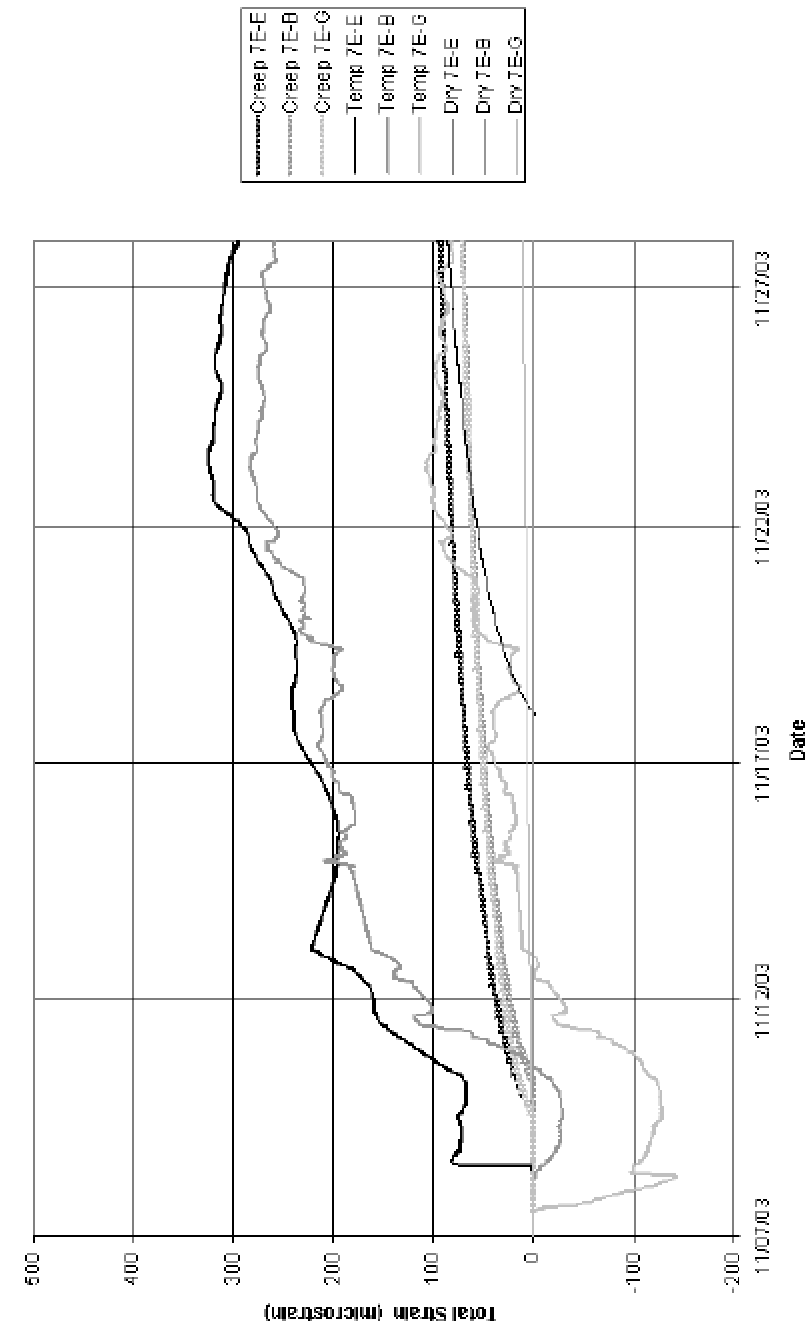

Figure B-3: Comparison of Calculated Strains for Gauge Location 7E (Interval 1).

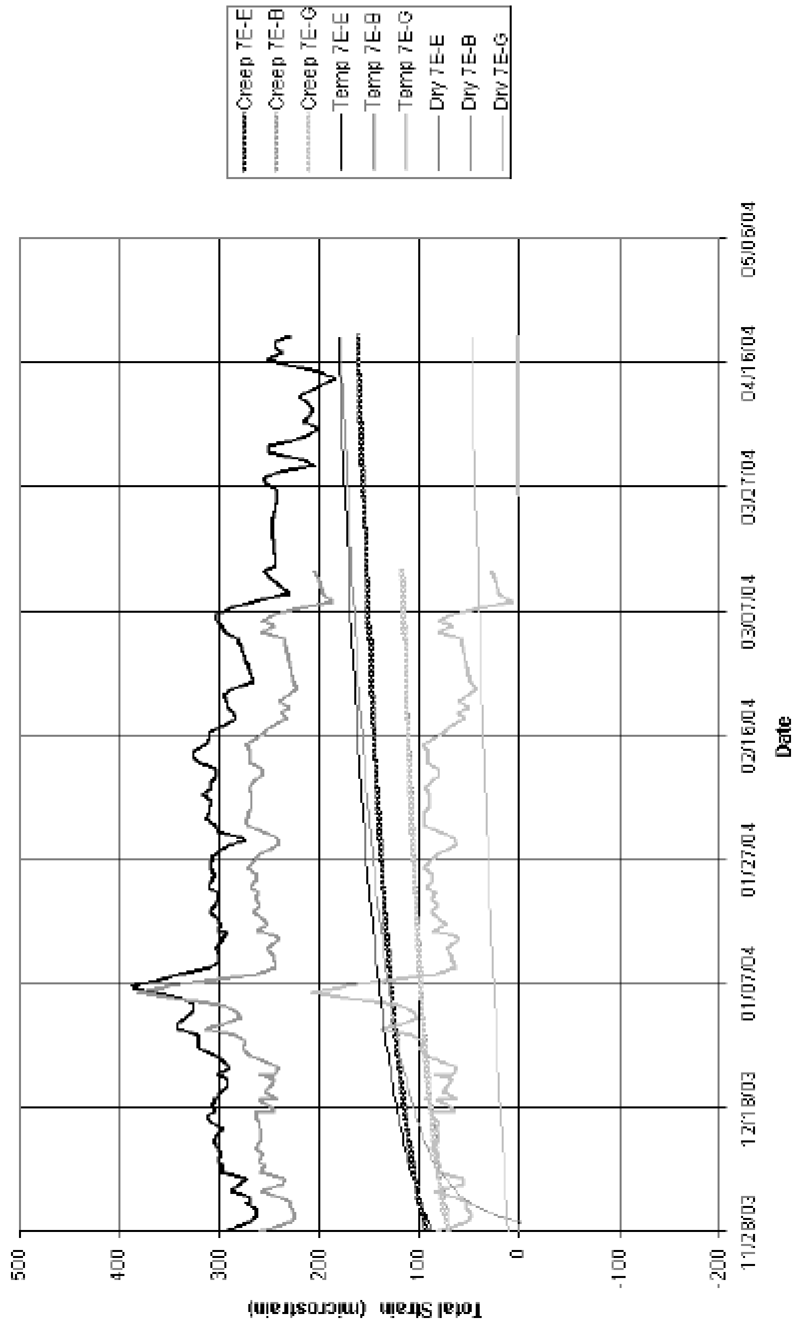

Figure B-4: Comparison of Calculated Strains for Gauge Location 7E (Interval 2).

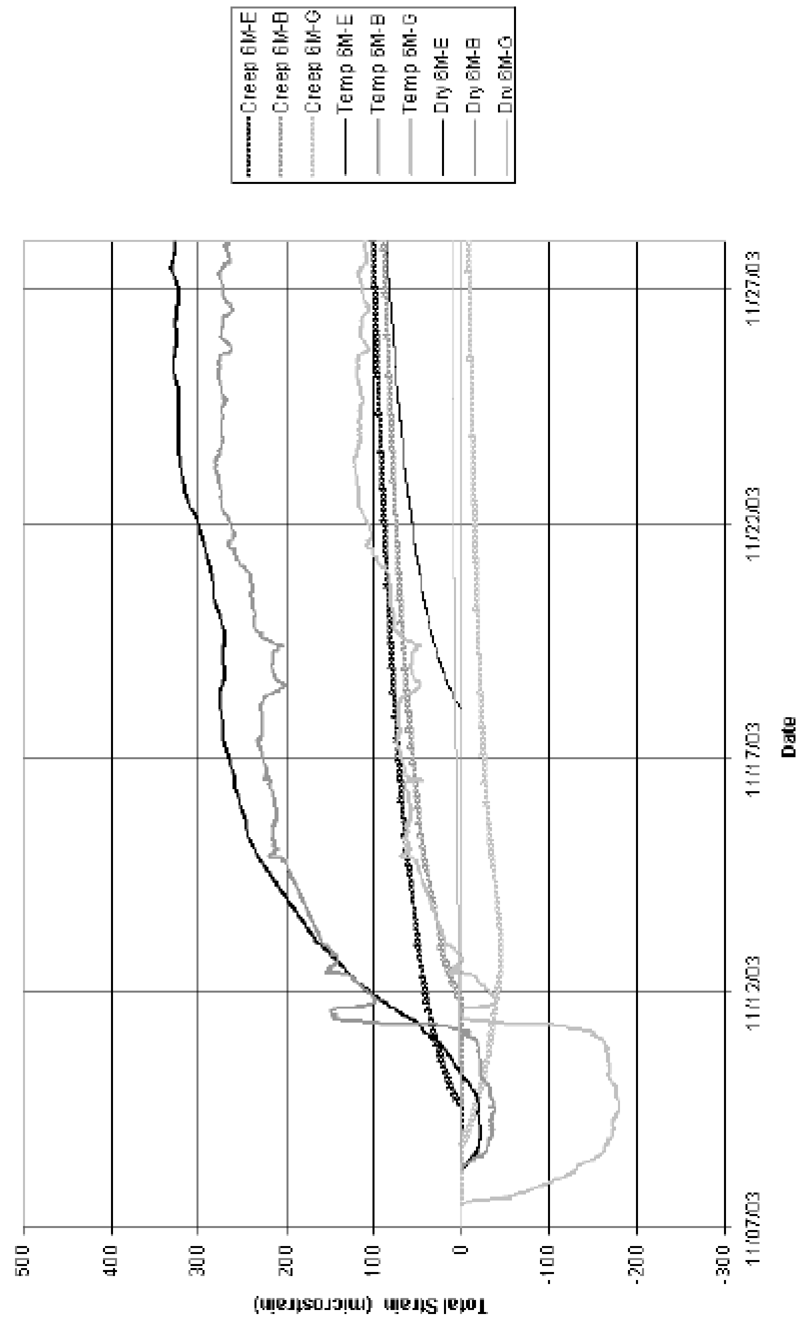

Figure B-5: Comparison of Calculated Strains for Gauge Location 6M (Interval 1).

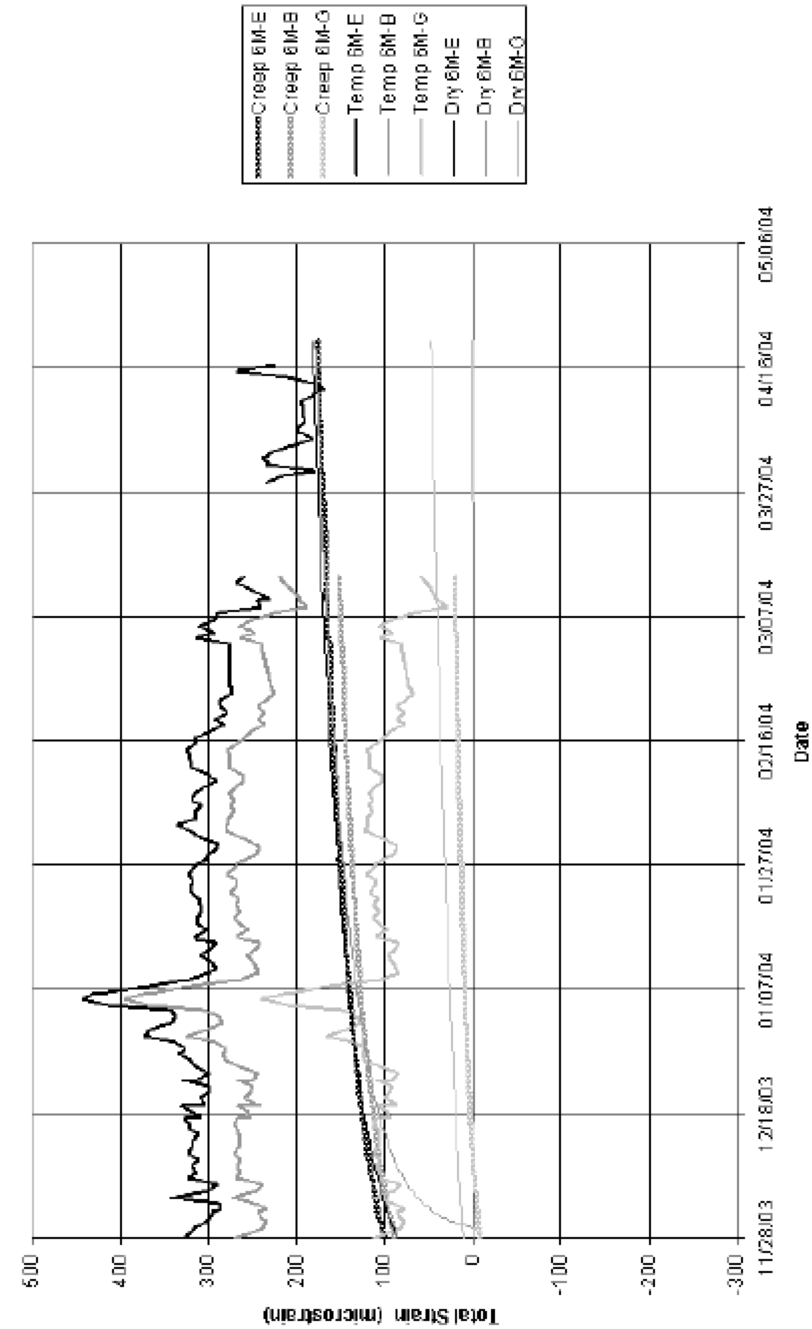

Figure B-6: Comparison of Calculated Strains for Gauge Location 6M (Interval 2).

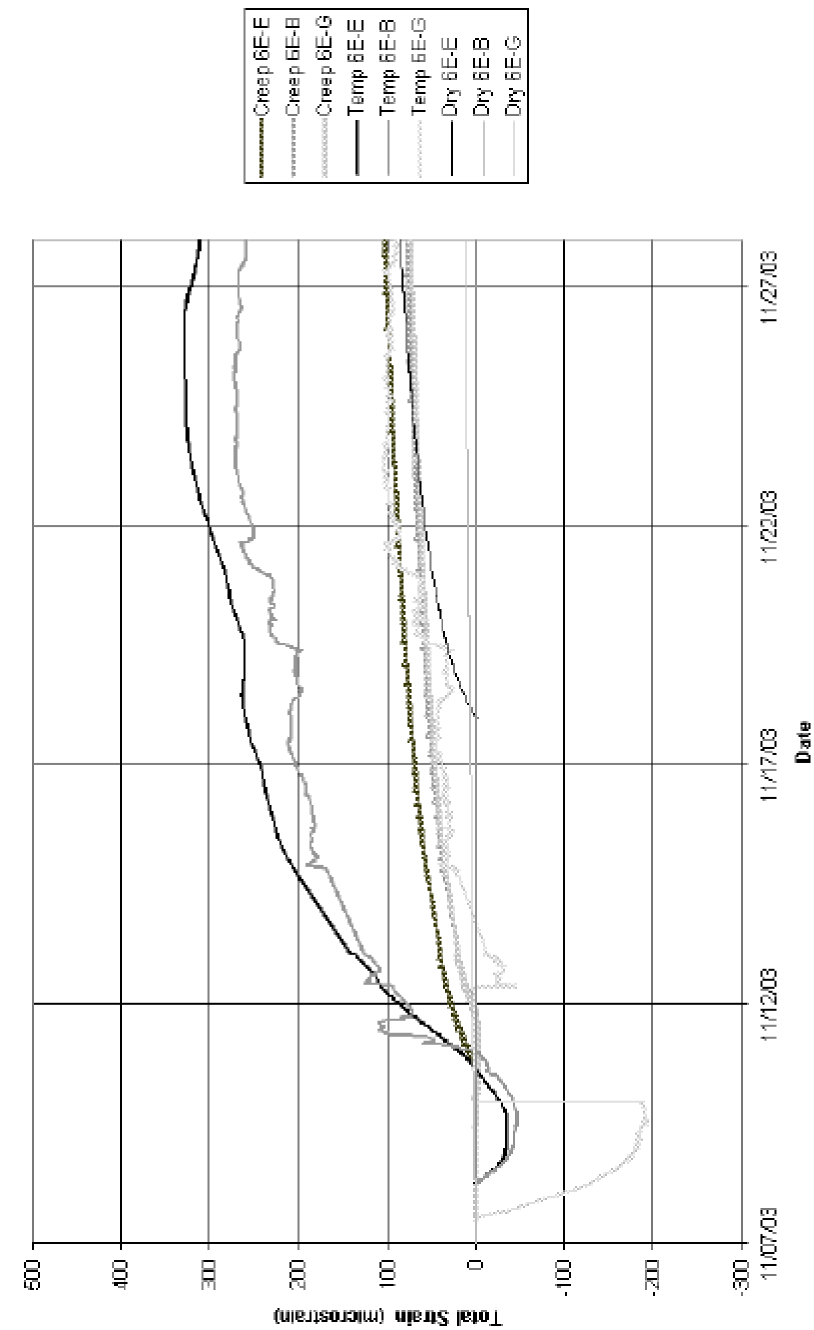

Figure B-7: Comparison of Calculated Strains for Gauge Location 6E (Interval 1).

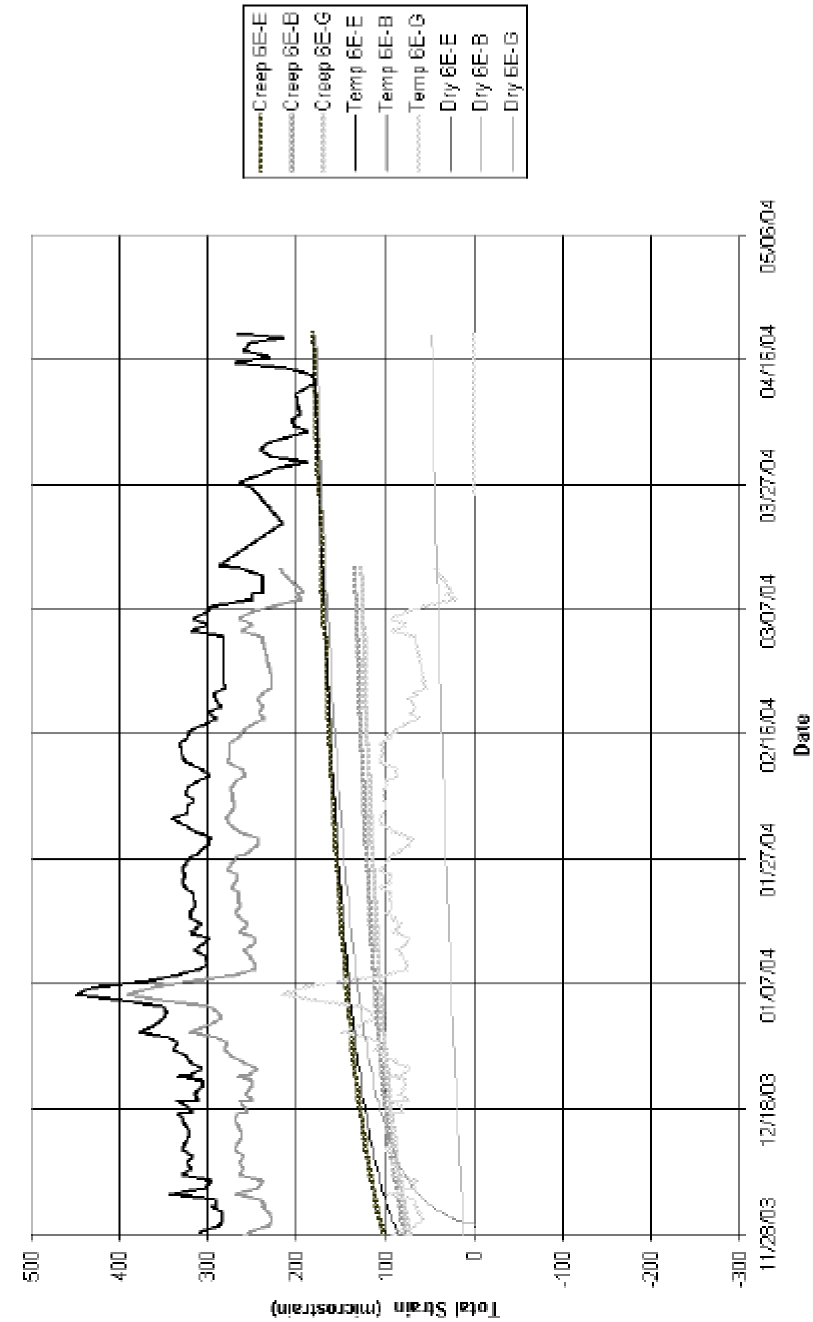

Figure B-8: Comparison of Calculated Strains for Gauge Location 6E (Interval 2).

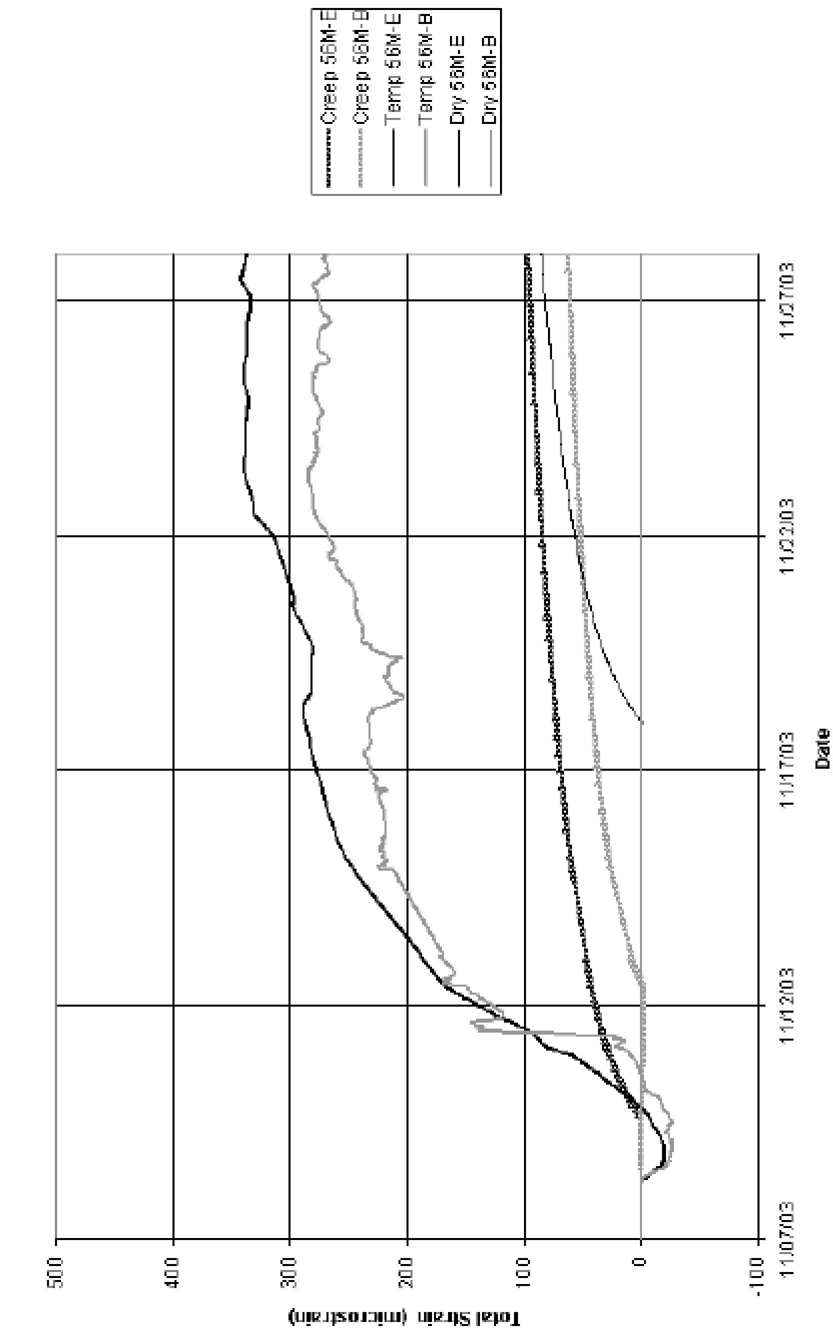

Figure B-9: Comparison of Calculated Strains for Gauge Location 56M (Interval 1).

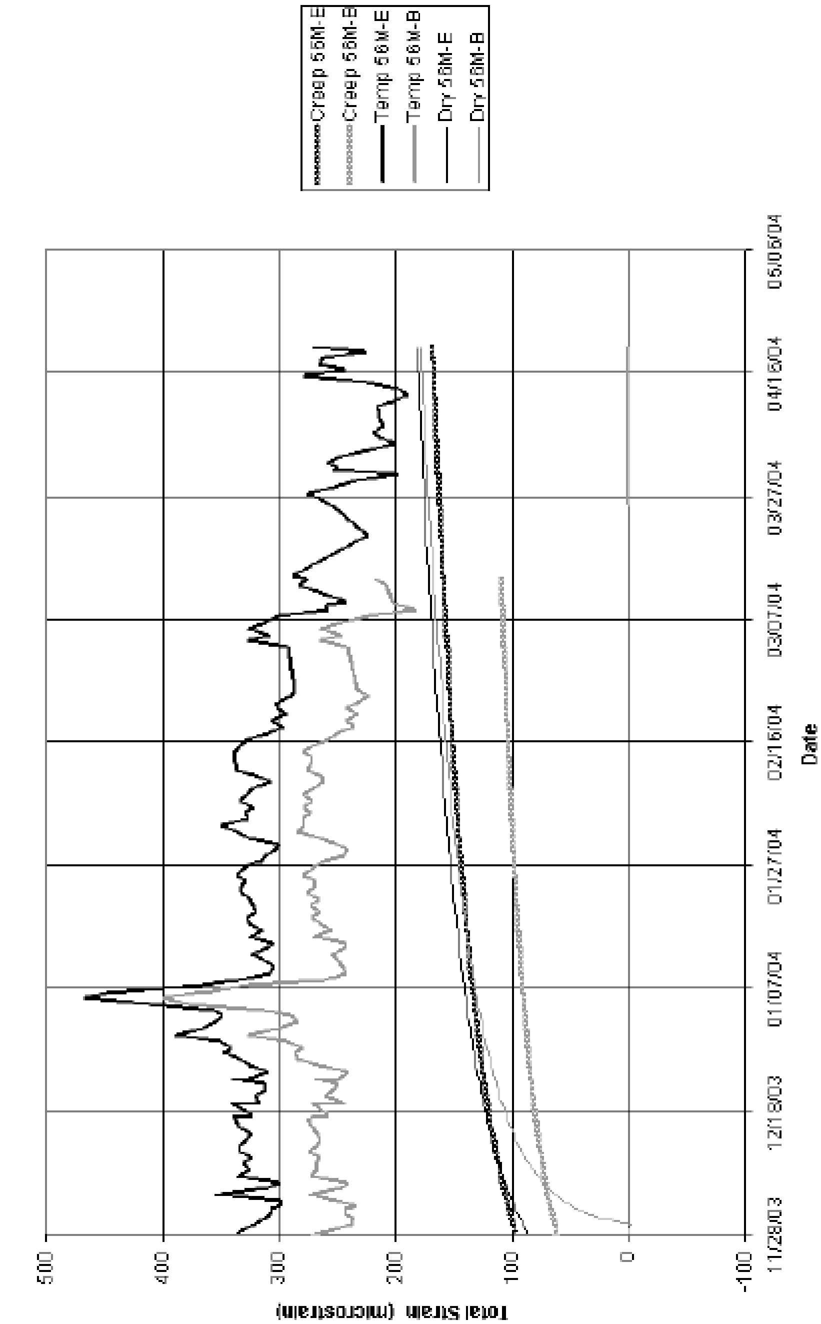

Figure B-10: Comparison of Calculated Strains for Gauge Location 56M (Interval 2).

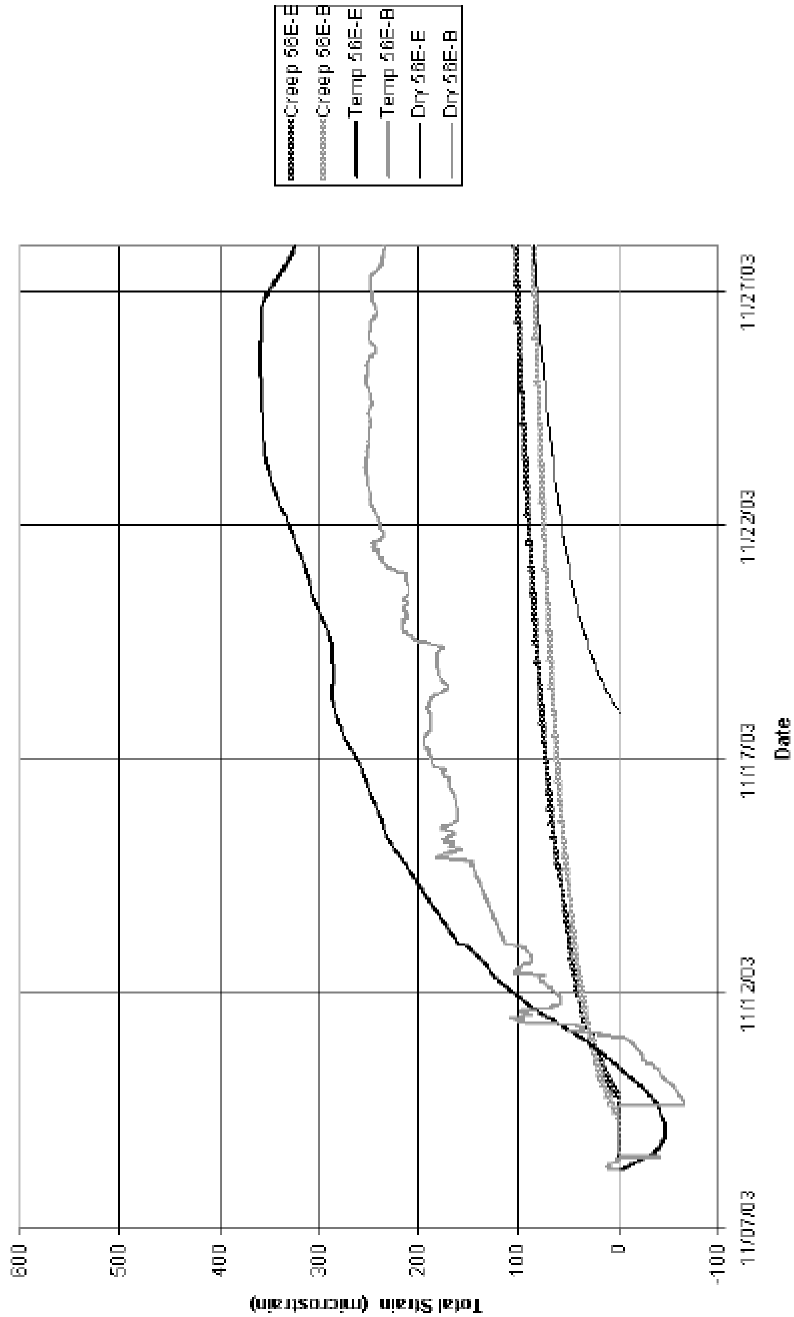

Figure B-11: Comparison of Calculated Strains for Gauge Location 56E (Interval 1).

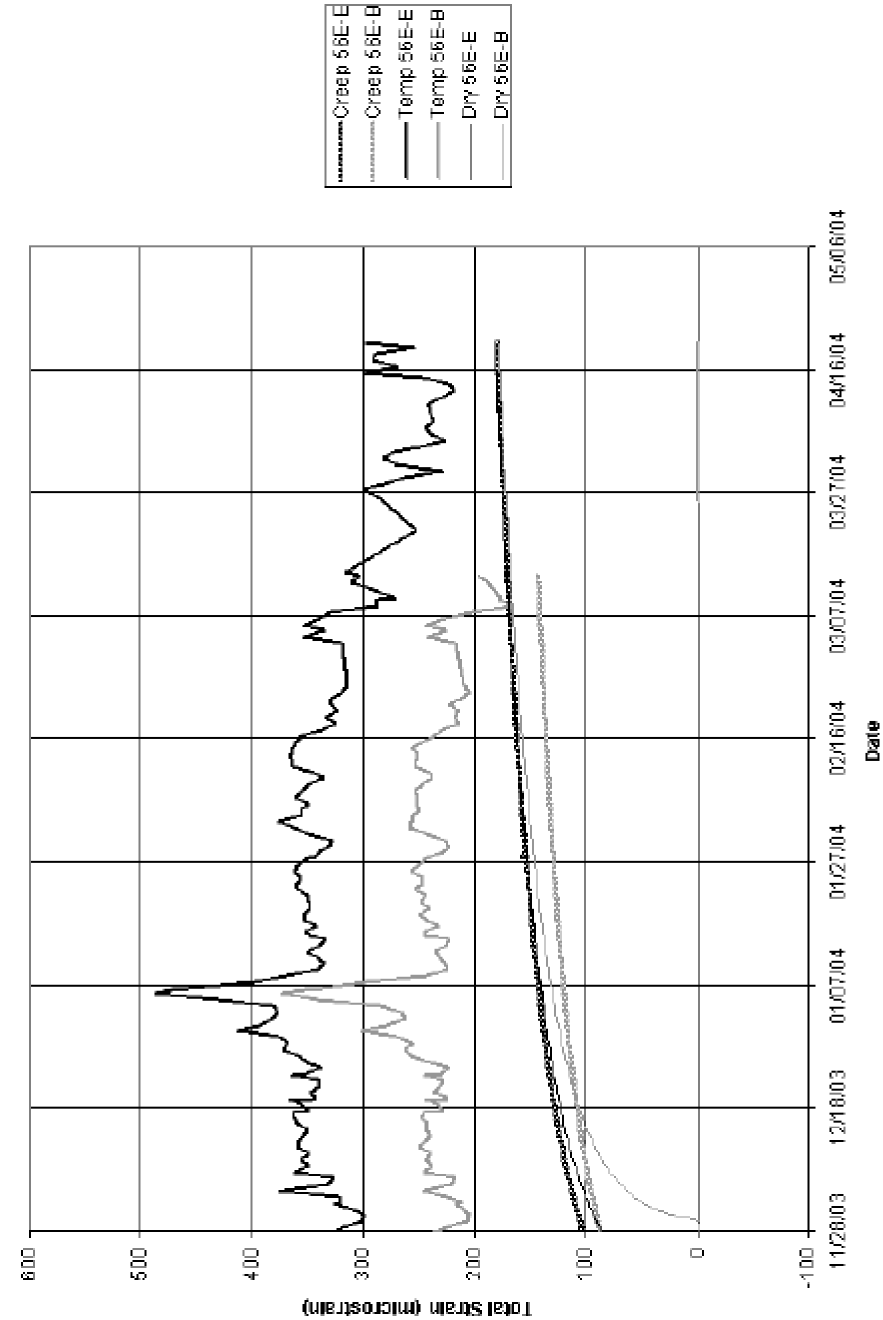

Figure B-12: Comparison of Calculated Strains for Gauge Location 56E (Interval 2).

APPENDIX C.   STRAIN DIFFERENCE GRAPHS

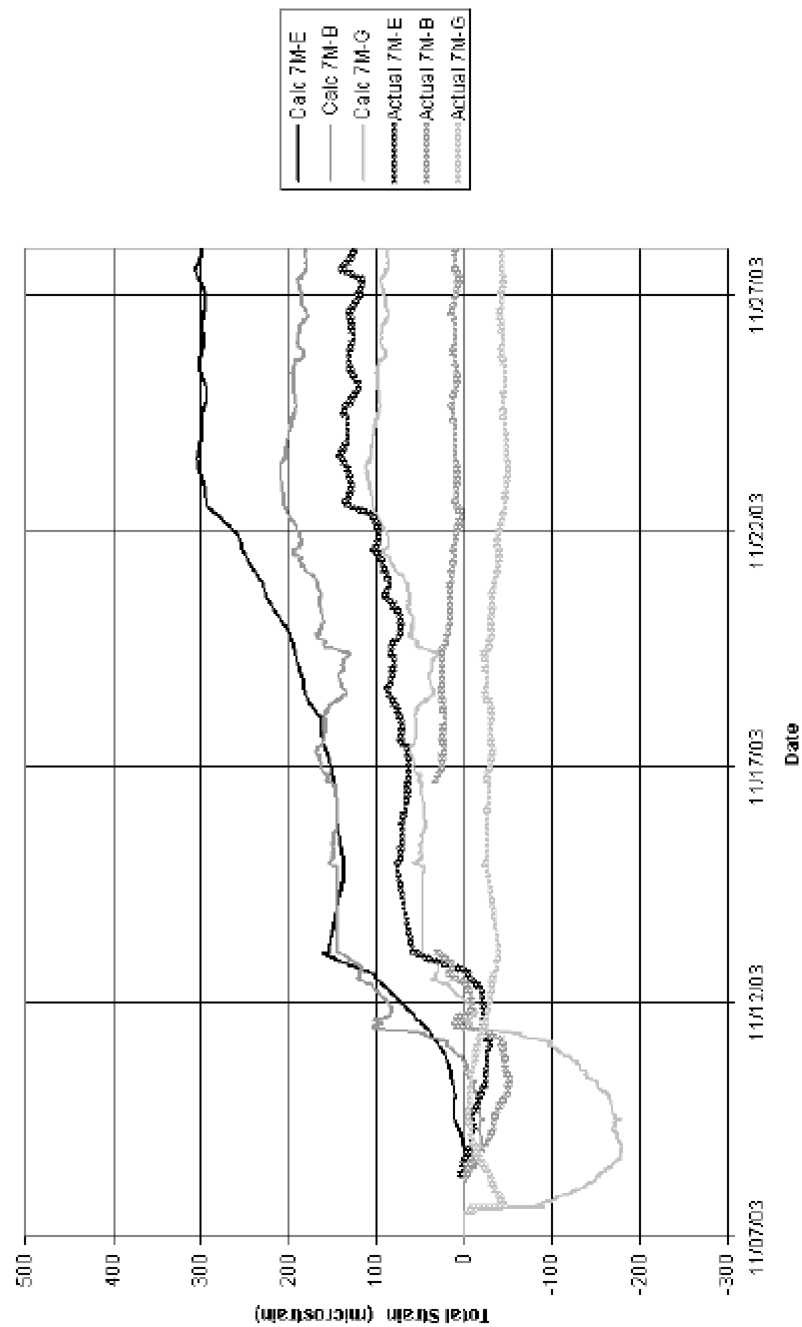

Figure C-1: Calculated vs. Actual Strain at Gauge Location 7M (Interval 1).

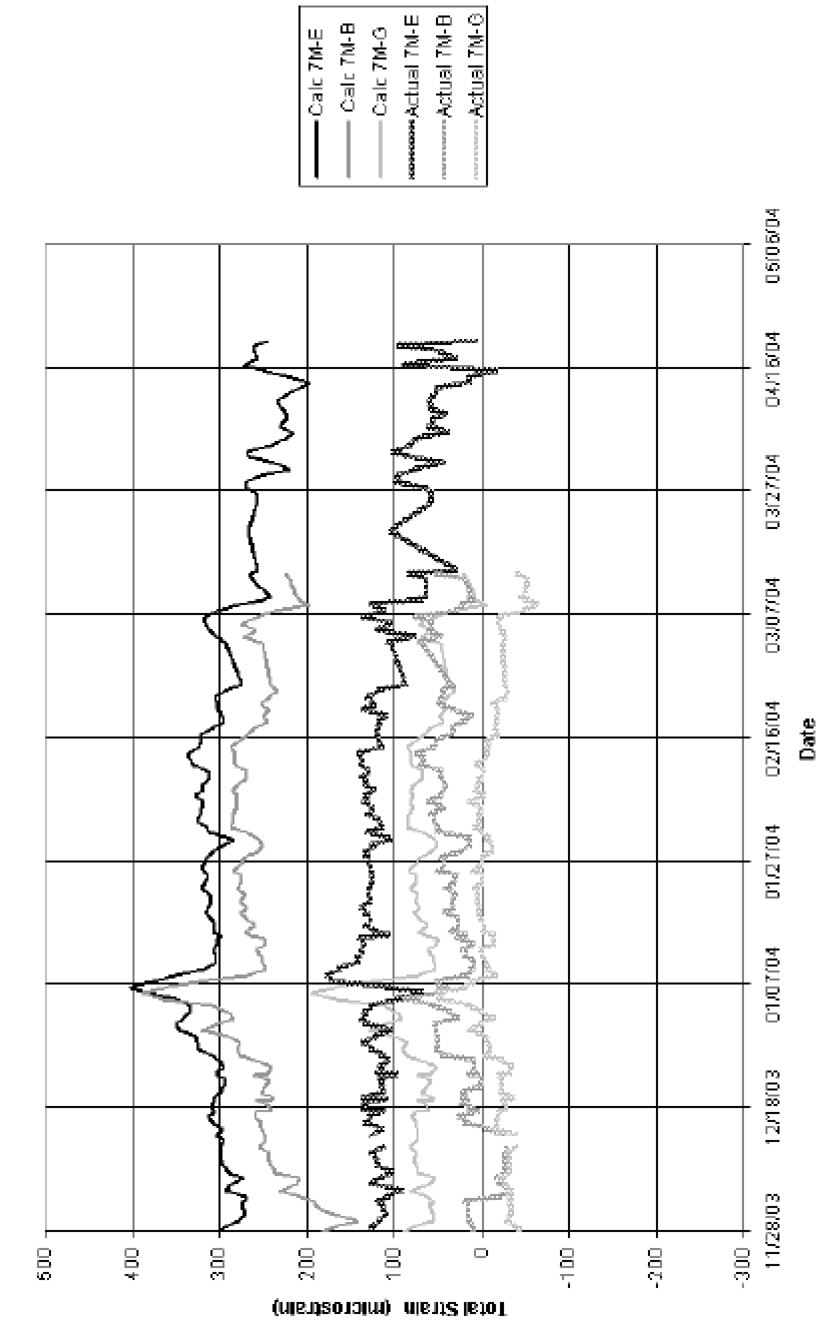

Figure C-2: Calculated vs. Actual Strain at Gauge Location 7M (Interval 2).

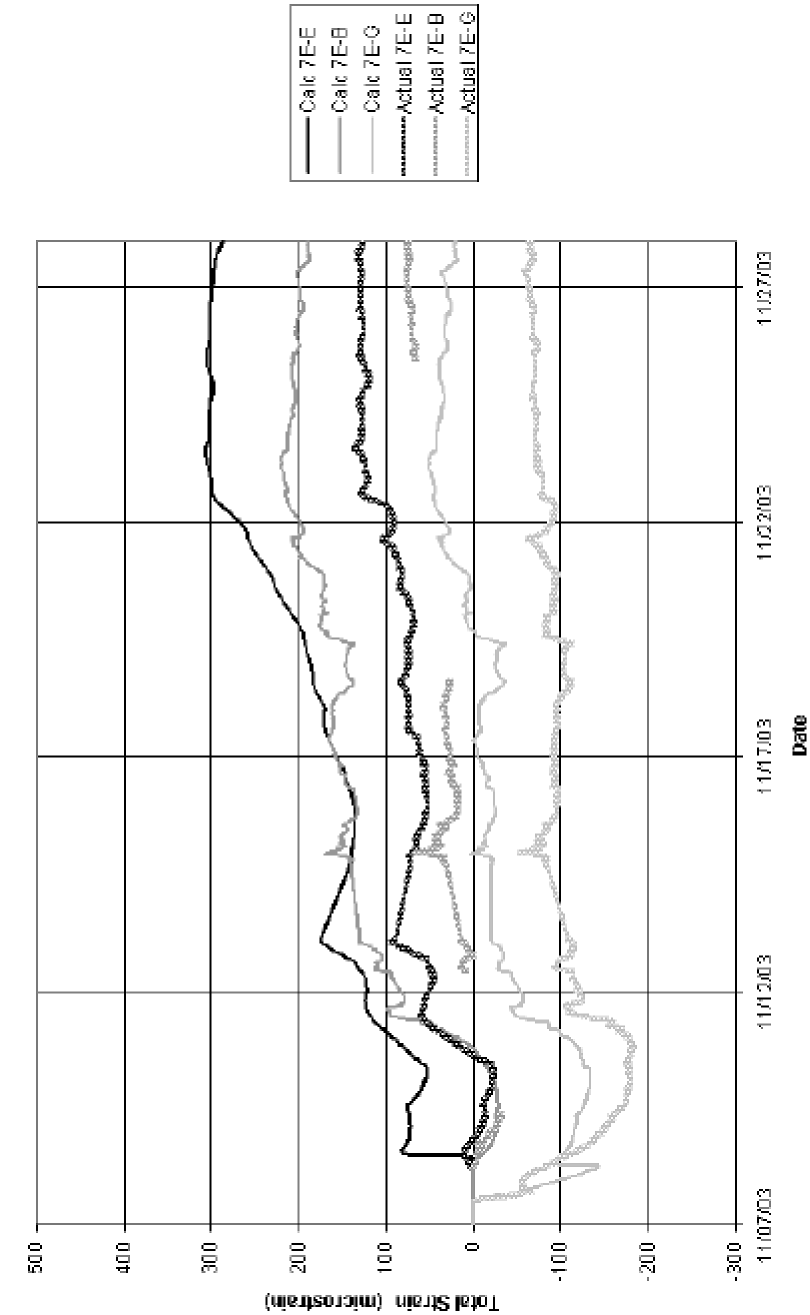

Figure C-3: Calculated vs. Actual Strain at Gauge Location 7E (Interval 1).

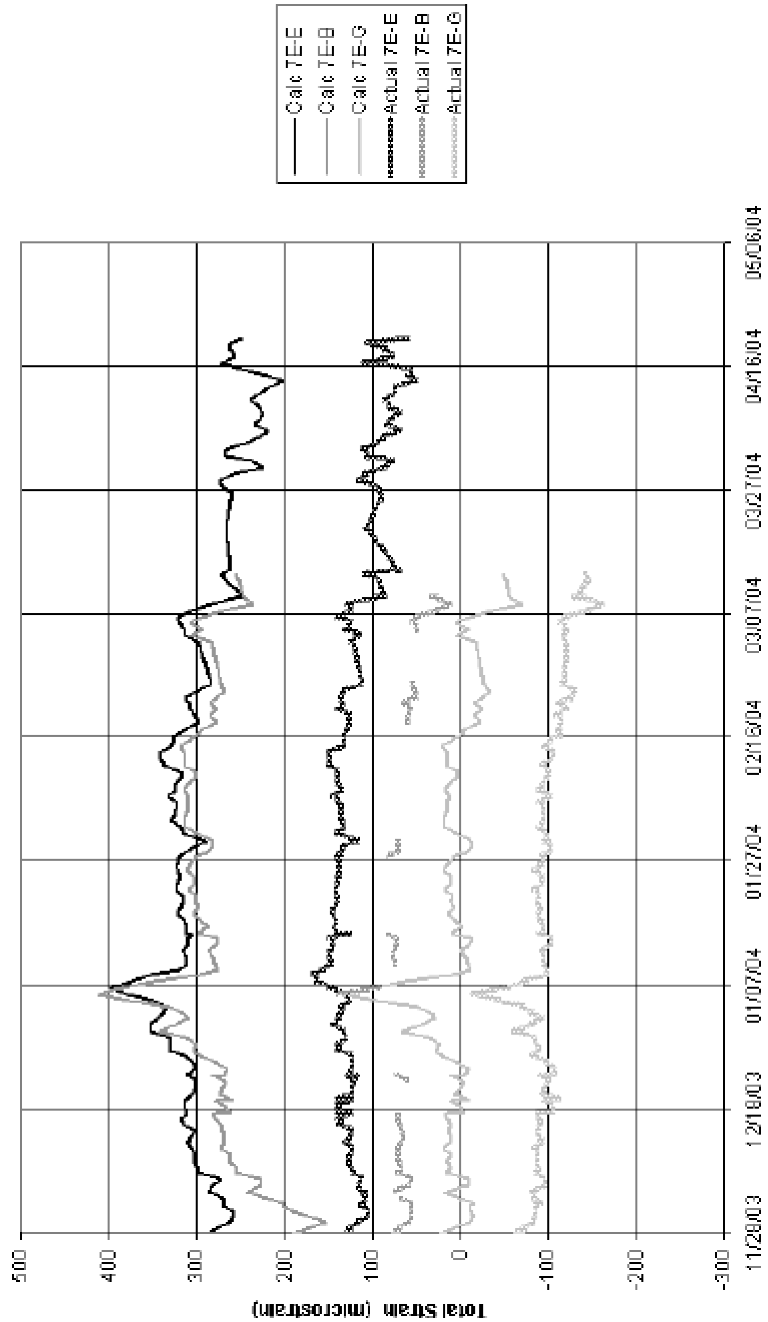

Figure C-4: Calculated vs. Actual Strain at Gauge Location 7E (Interval 2).

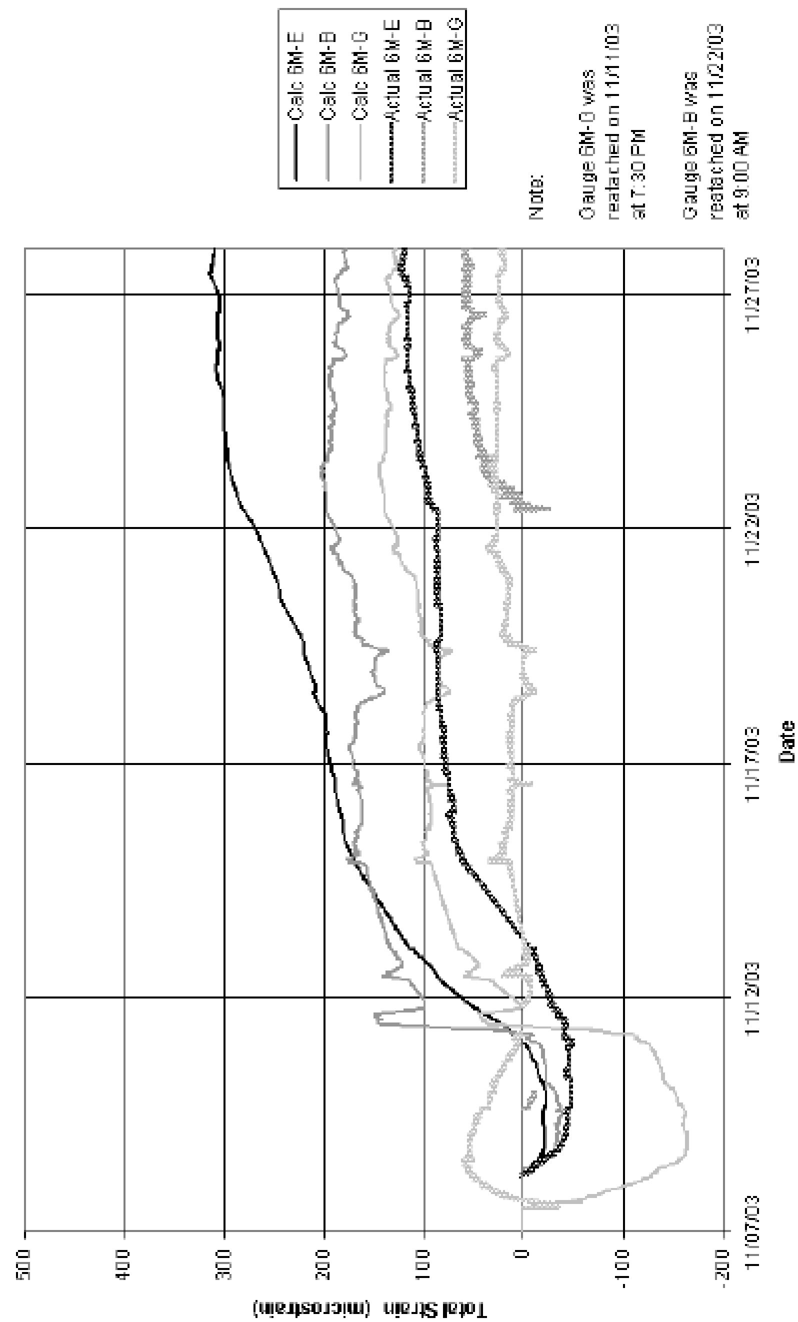

Figure C-5: Calculated vs. Actual Strain at Gauge Location 6M (Interval 1).

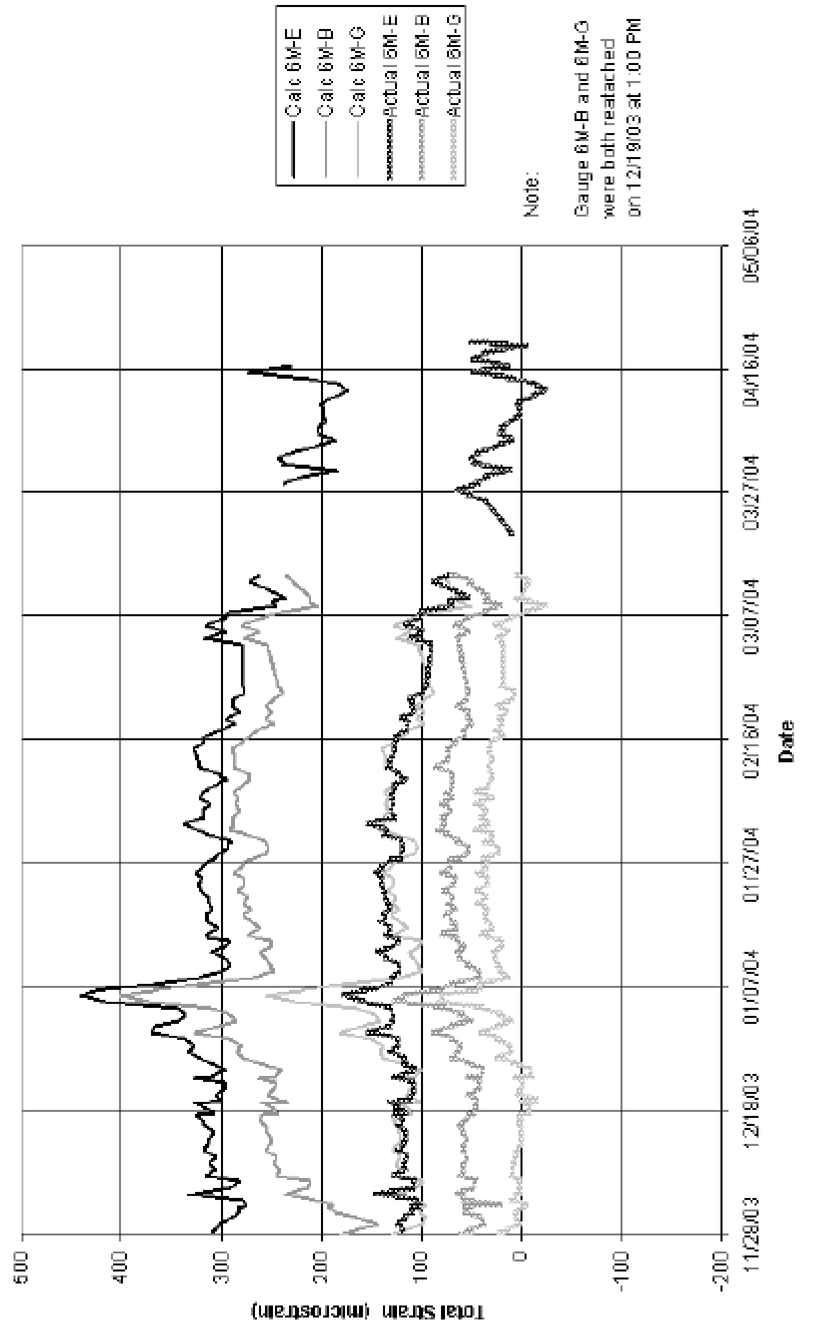

Figure C-6: Calculated vs. Actual Strain at Gauge Location 6M (Interval 2).

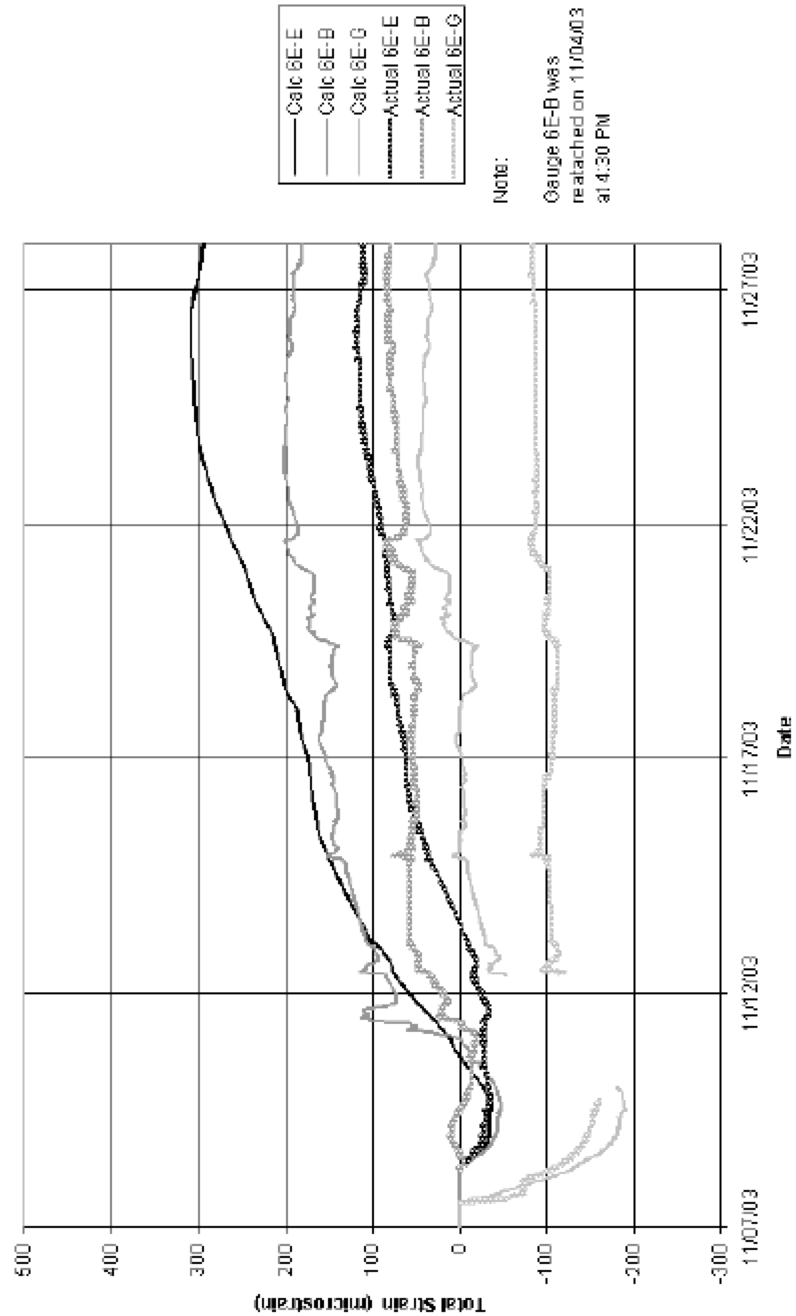

Figure C-7: Calculated vs. Actual Strain at Gauge Location 6E (Interval 1).

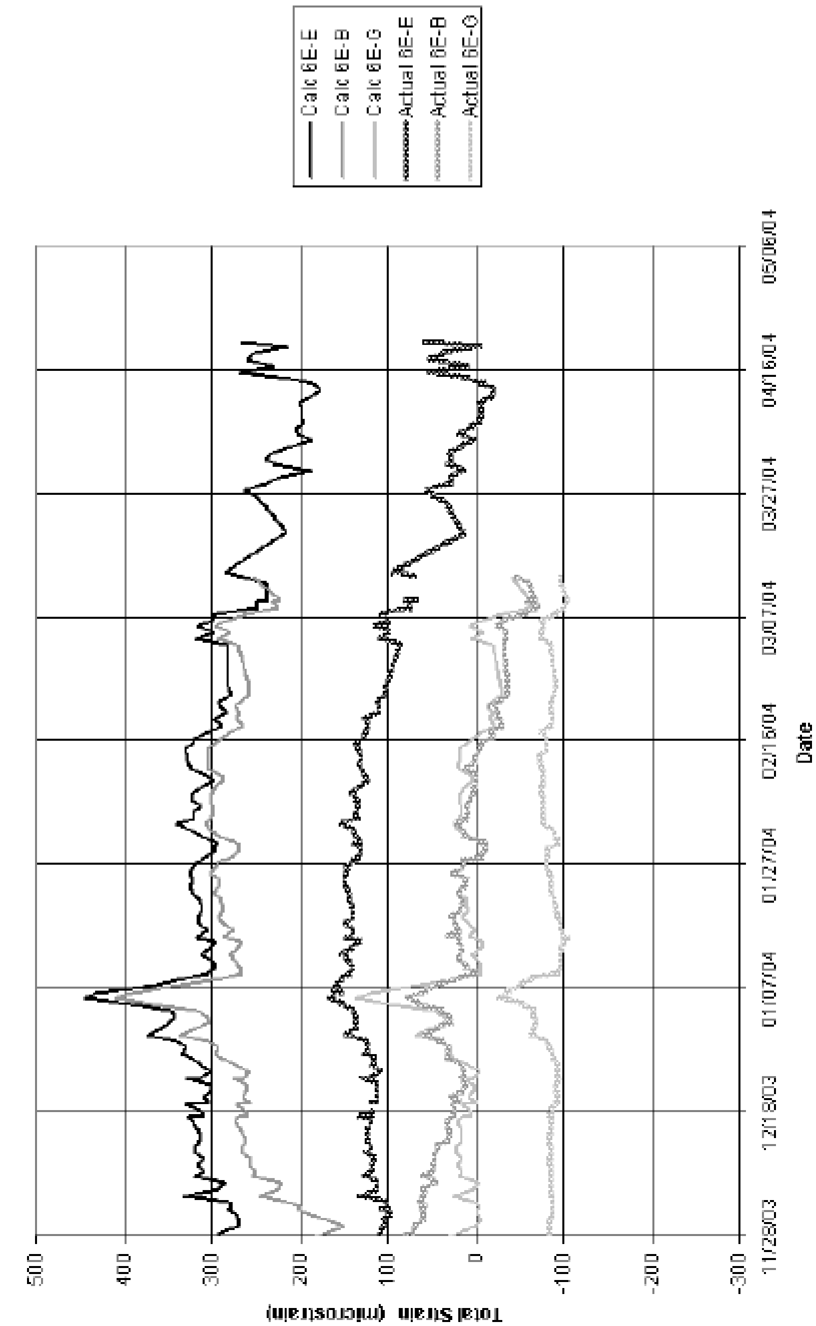

Figure C-8: Calculated vs. Actual Strain at Gauge Location 6E (Interval 2).

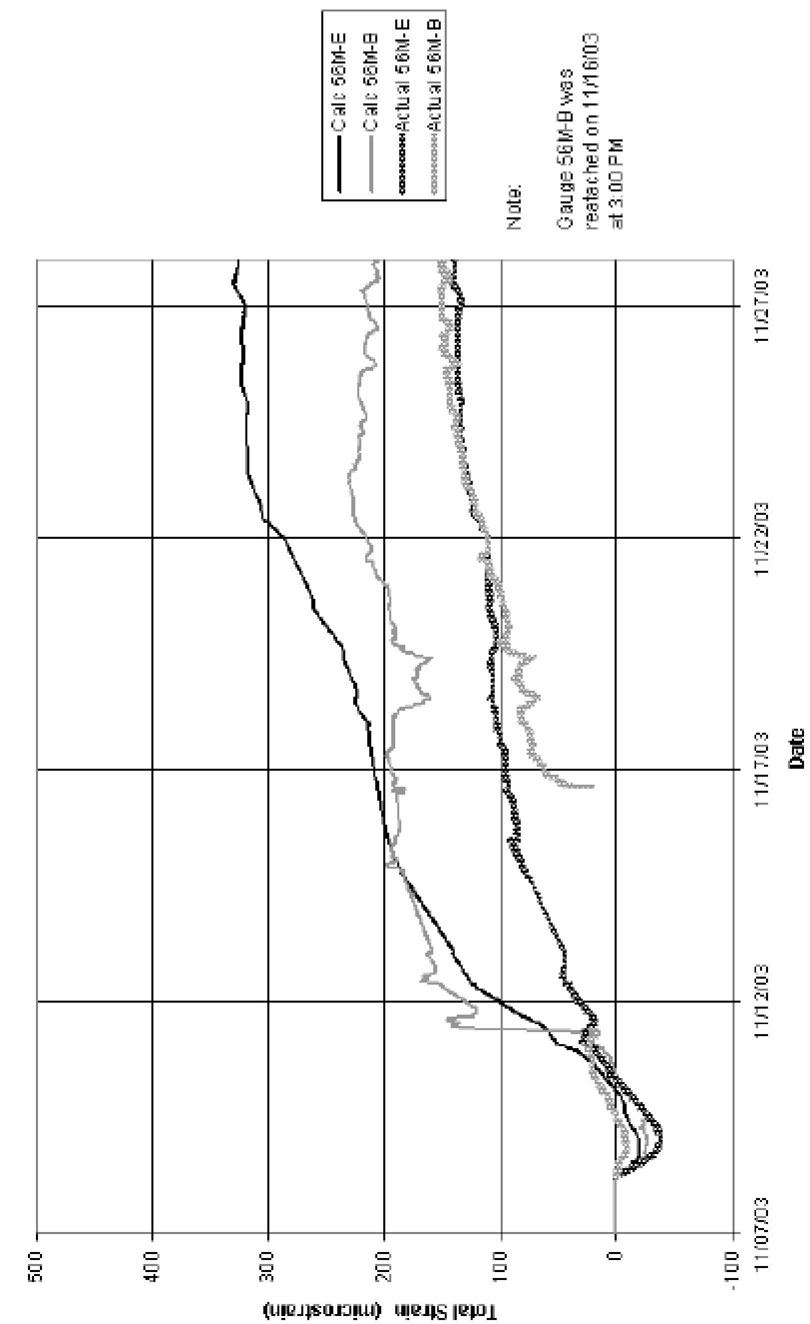

Figure C-9: Calculated vs. Actual Strain at Gauge Location 56M (Interval 1).

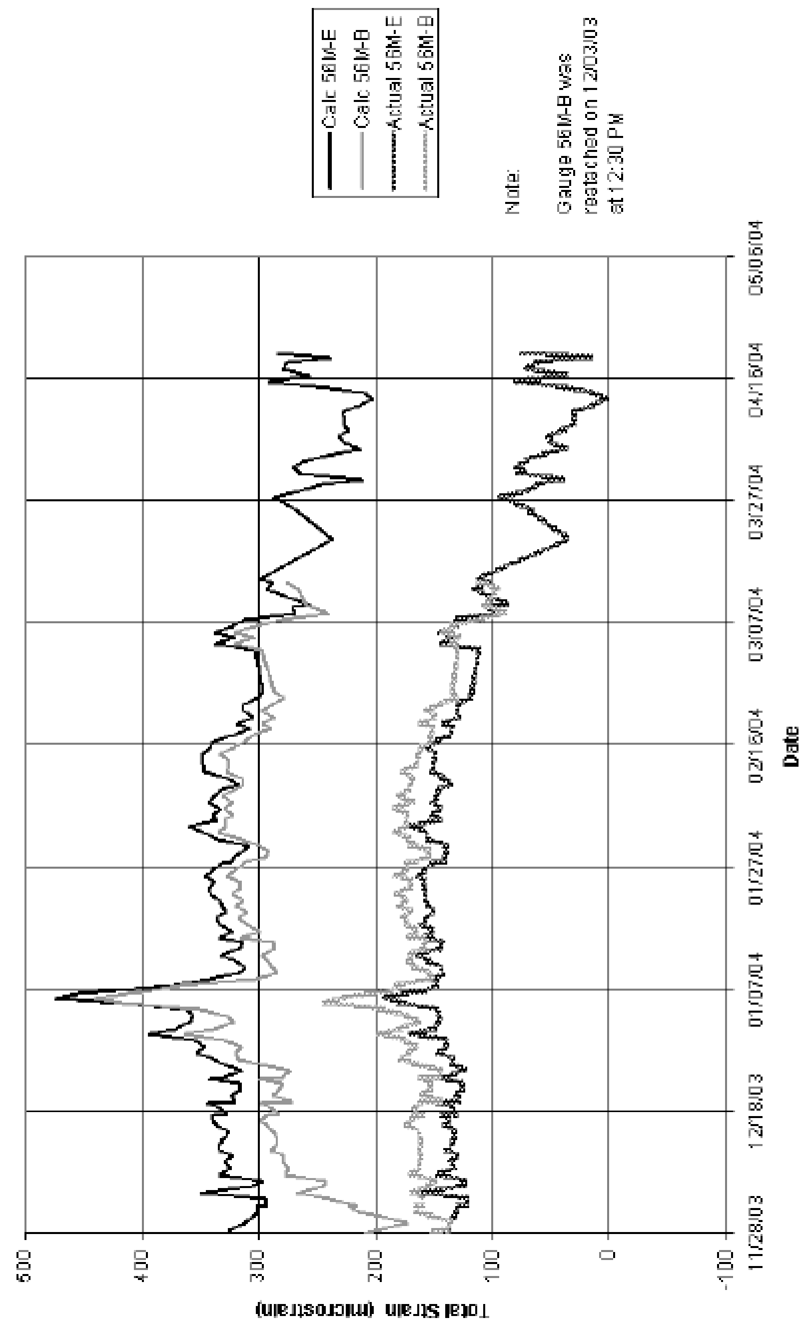

Figure C-10: Calculated vs. Actual Strain at Gauge Location 56M (Interval 2).

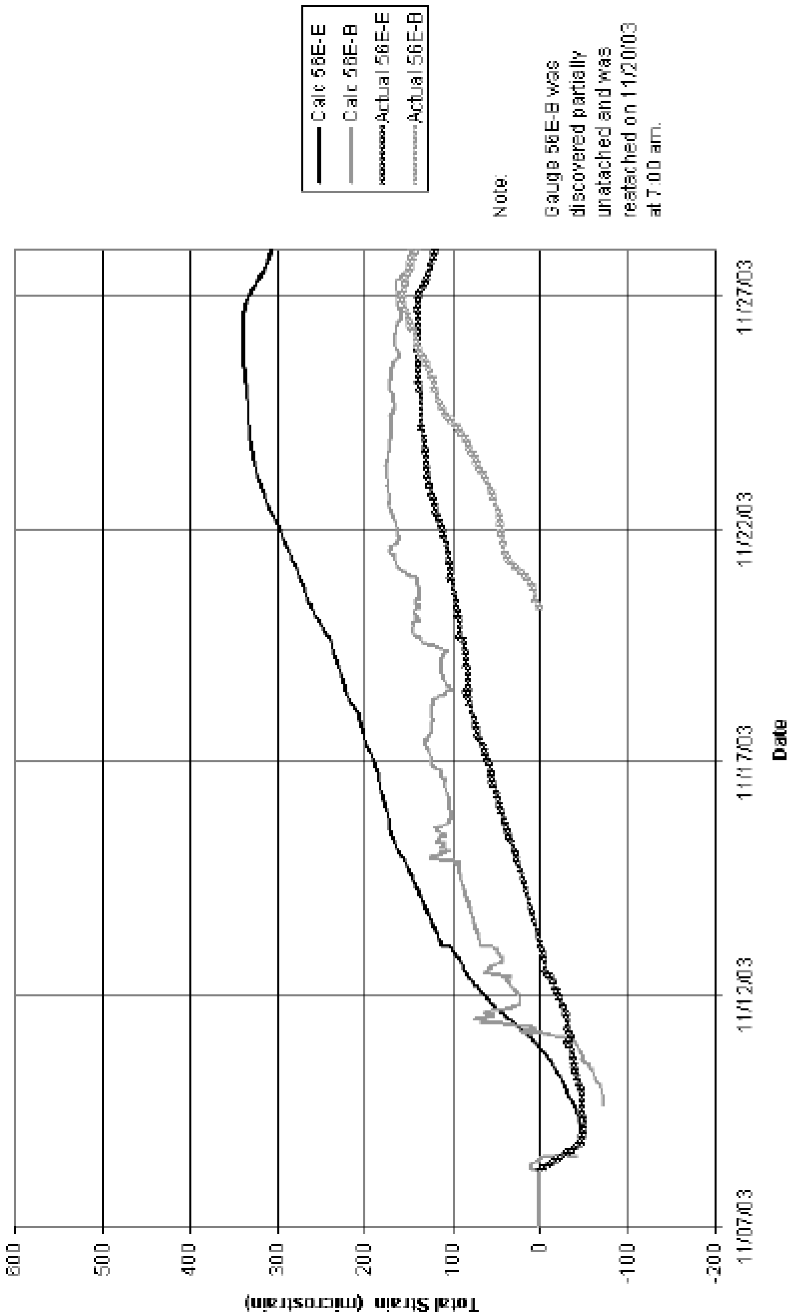

Figure C-11: Calculated vs. Actual Strain at Gauge Location 56E (Interval 1).

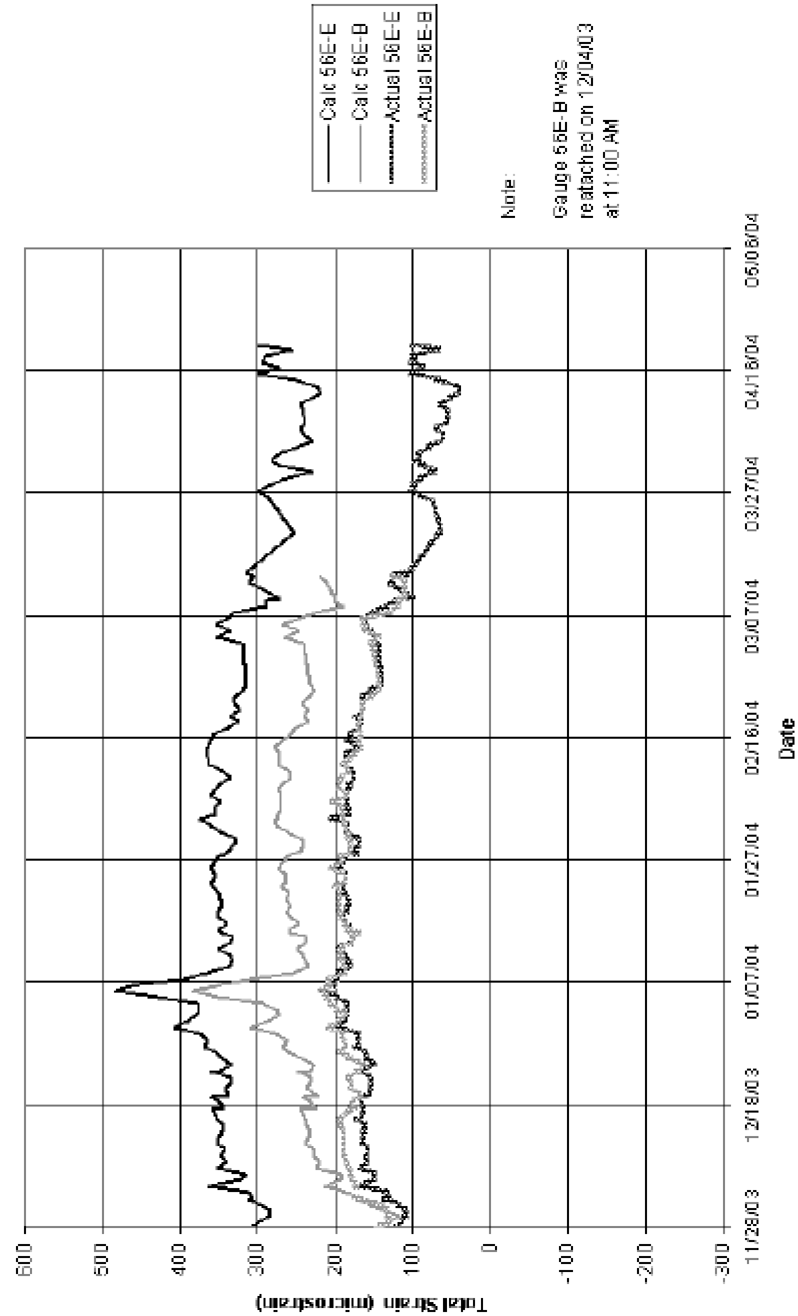

Figure C-12: Calculated vs. Actual Strain at Gauge Location 56E (Interval 2).

APPENDIX D. DECK AND GIRDER STRESS GRAPHS

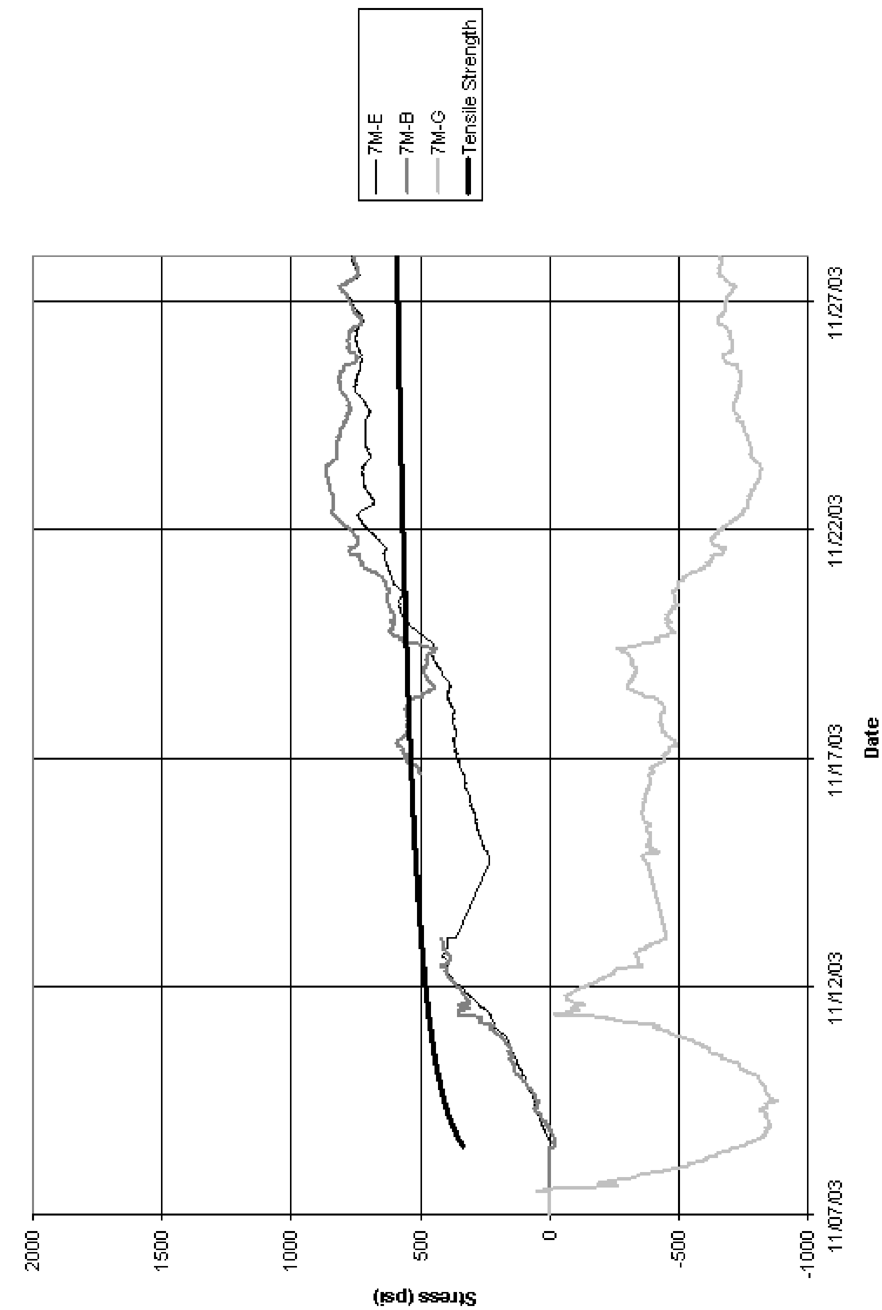

Figure D-1: Potential Deck Stresses at Gauge Location 7M (Interval 1).

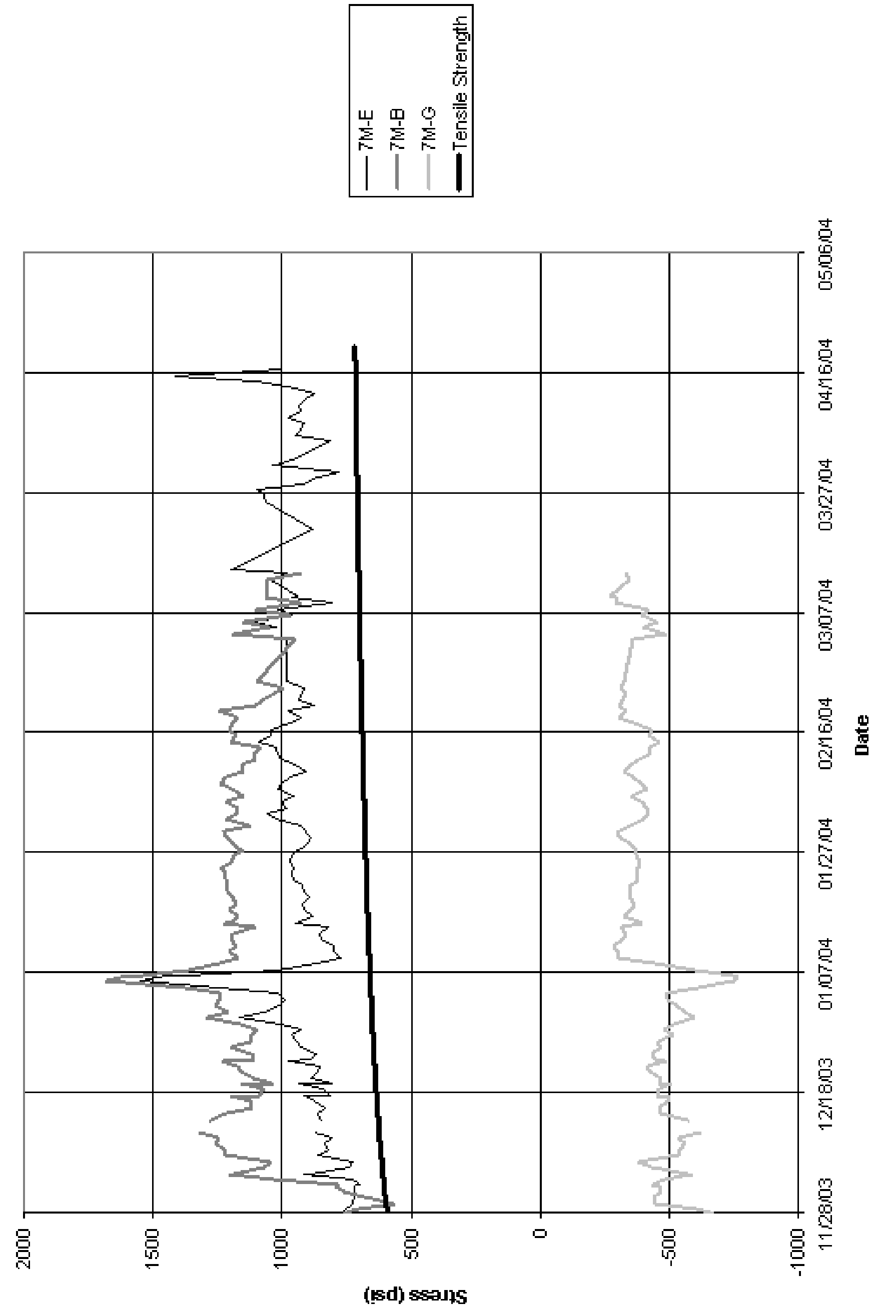

Figure D-2: Potential Deck Stresses at Gauge Location 7M (Interval 2).

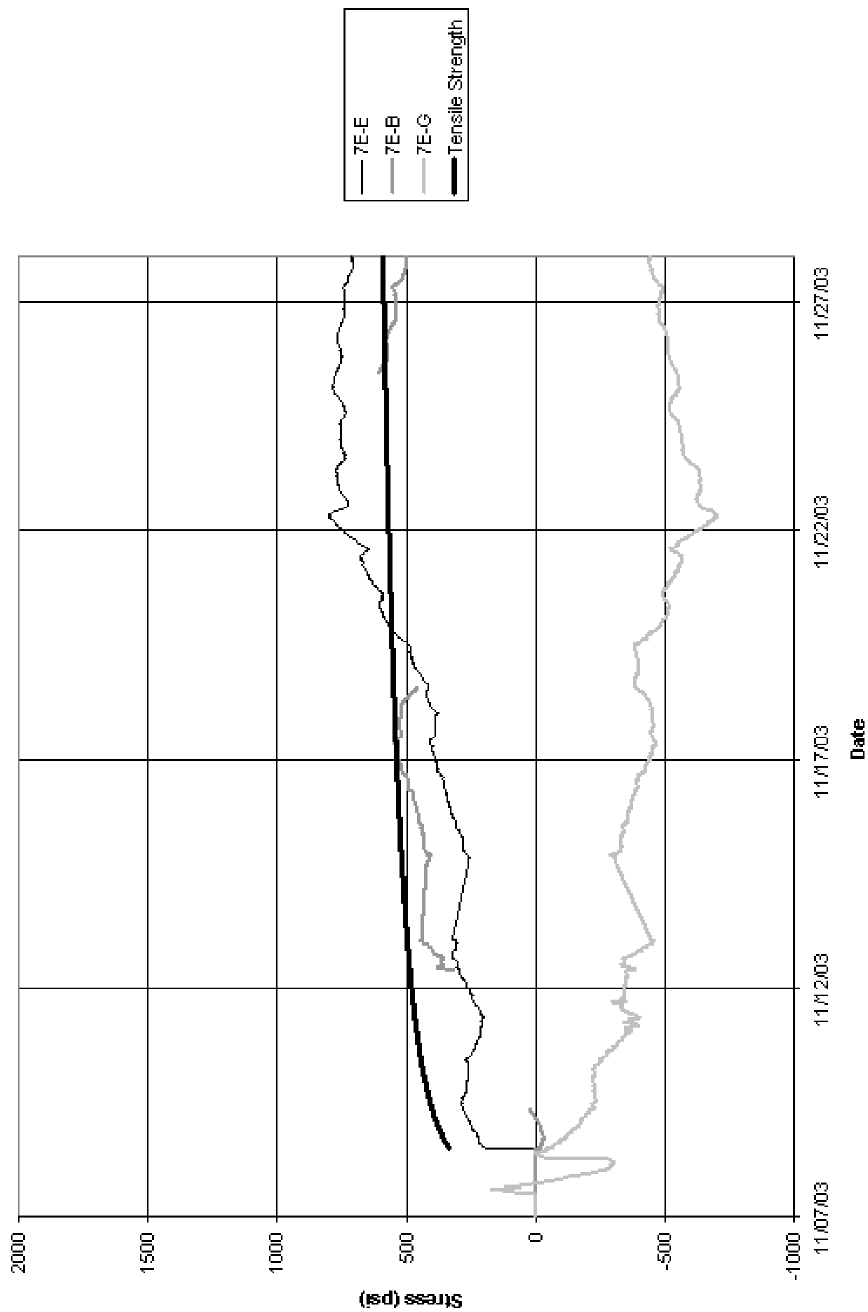

Figure D-3: Potential Deck Stresses at Gauge Location 7E (Interval 1).

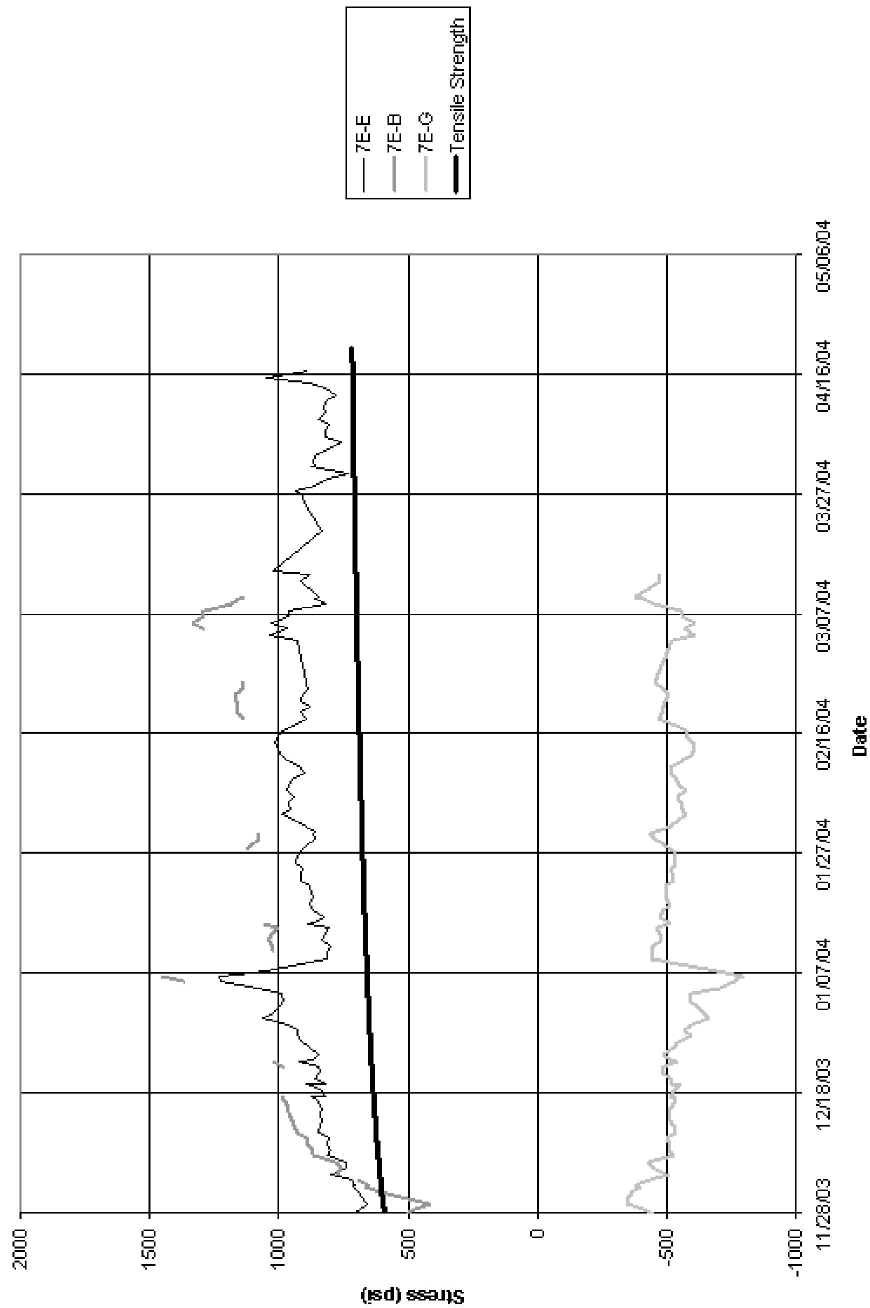

Figure D-4: Potential Deck Stresses at Gauge Location 7E (Interval 2).

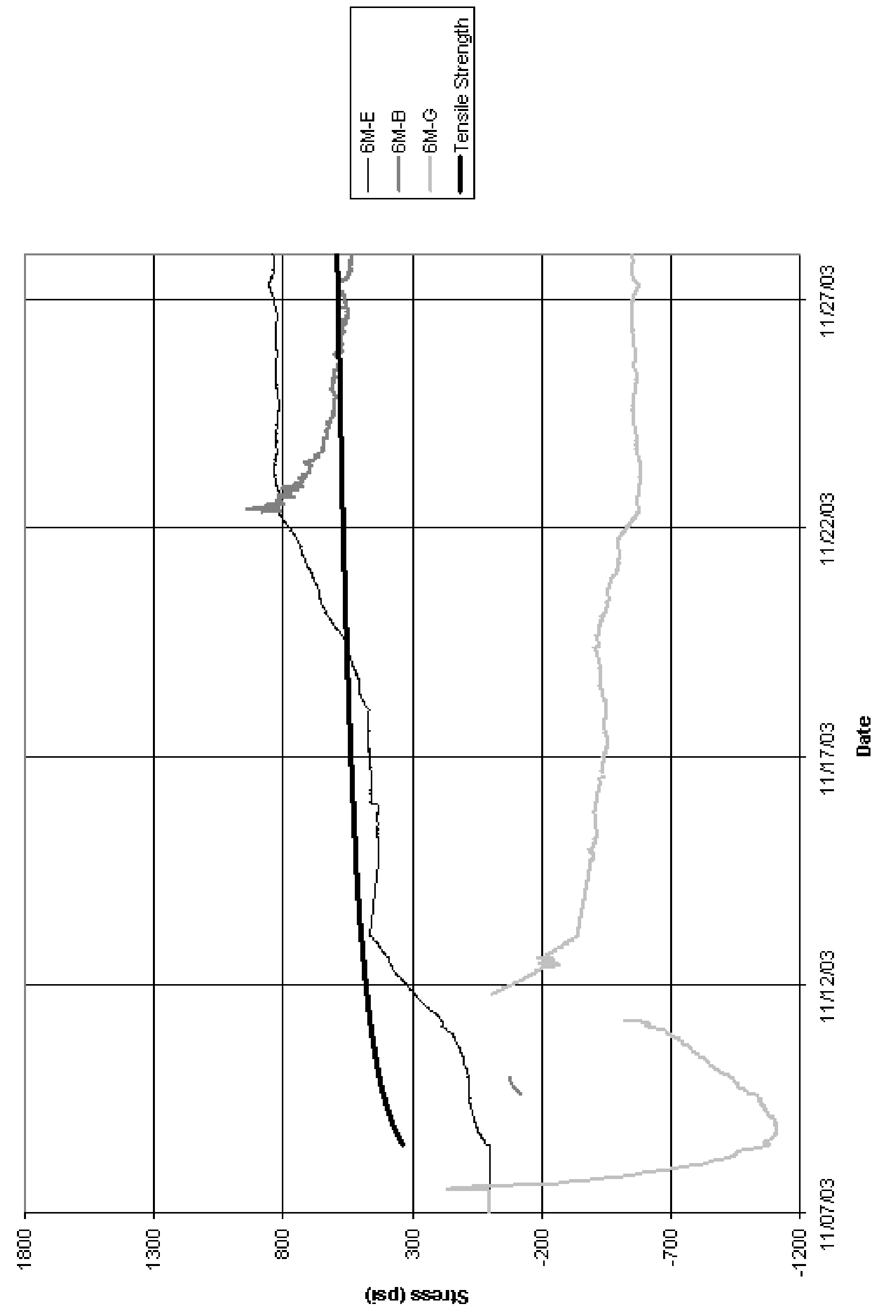

Figure D-5: Potential Deck Stresses at Gauge Location 6M (Interval 1).

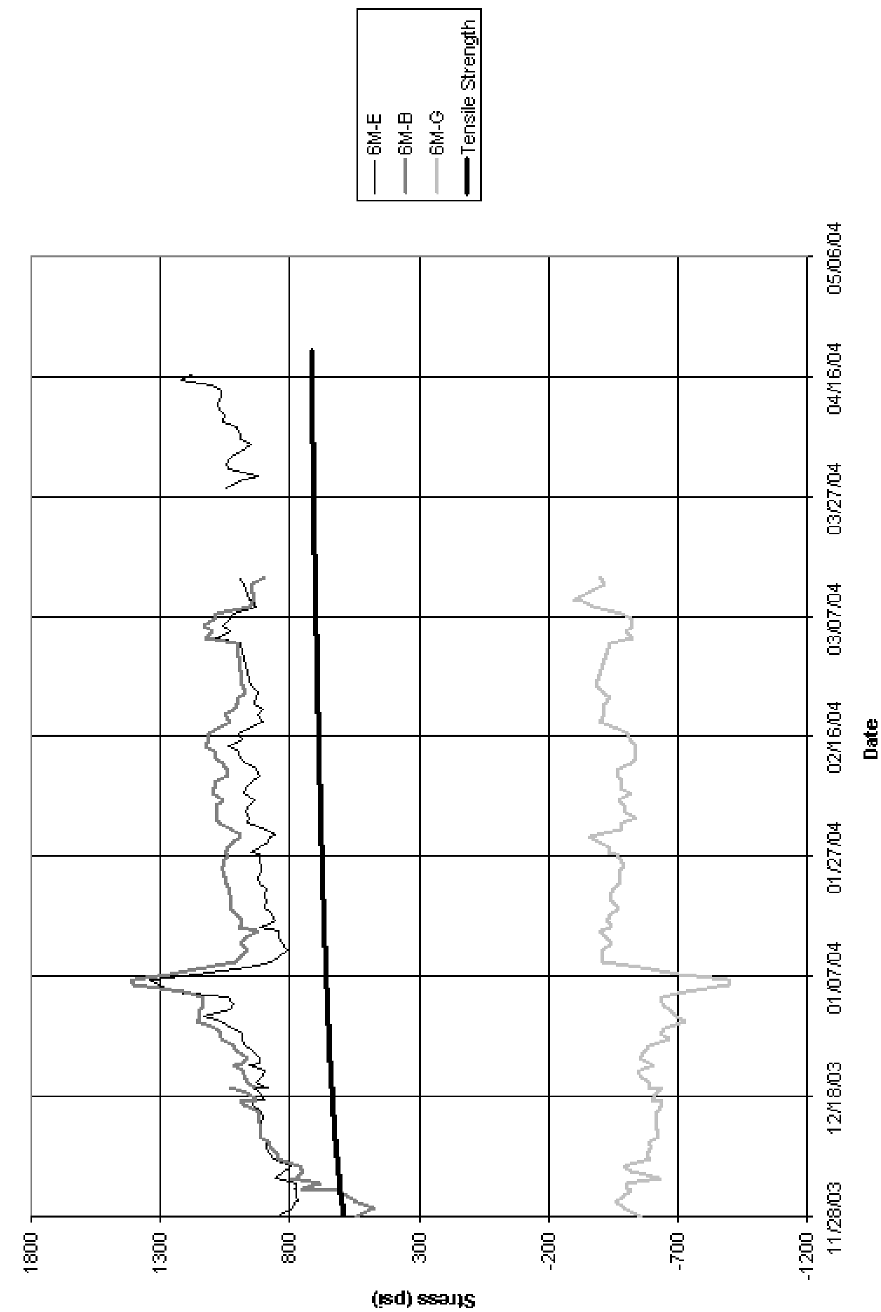

Figure D-6: Potential Deck Stresses at Gauge Location 6M (Interval 2).

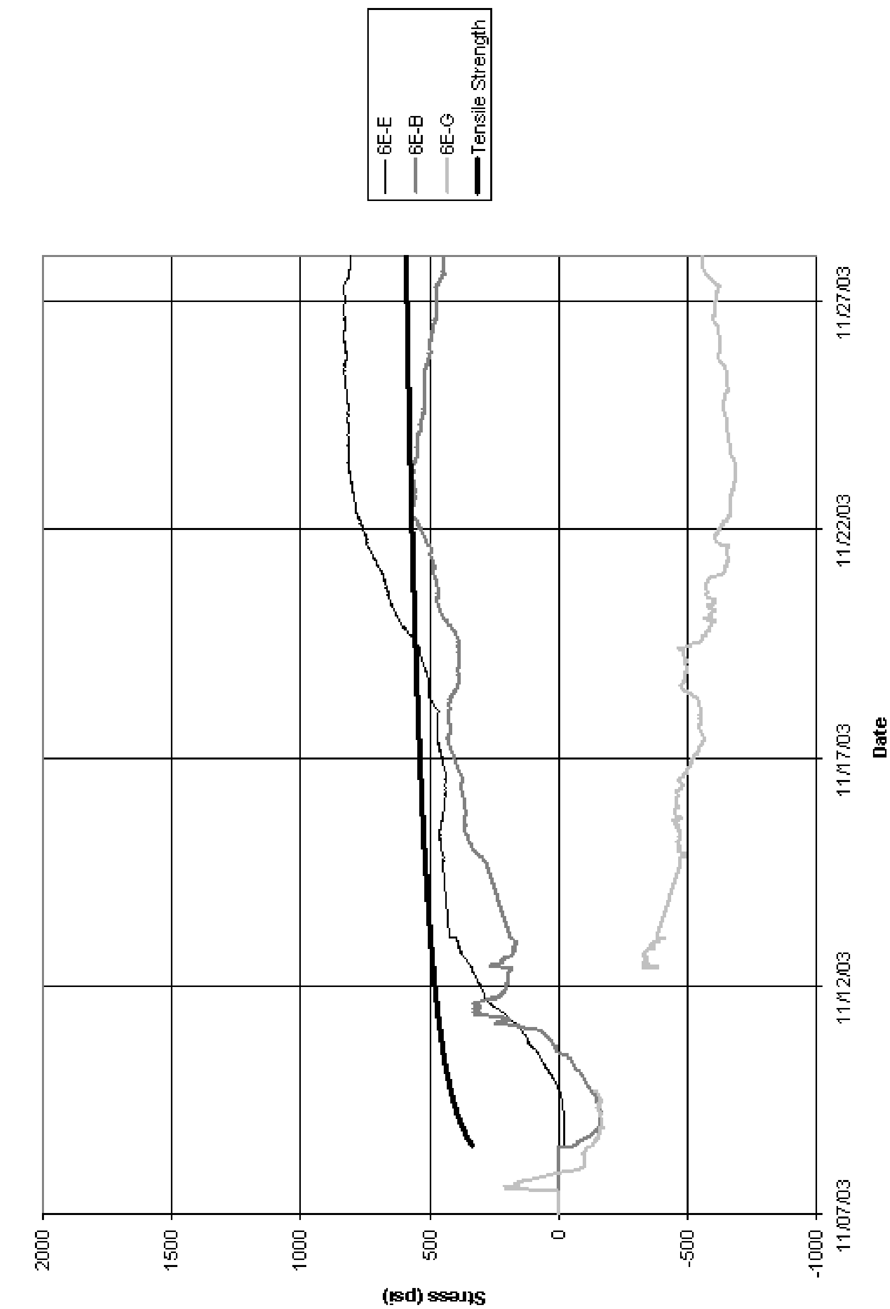

Figure D-7: Potential Deck Stresses at Gauge Location 6E (Interval 1).

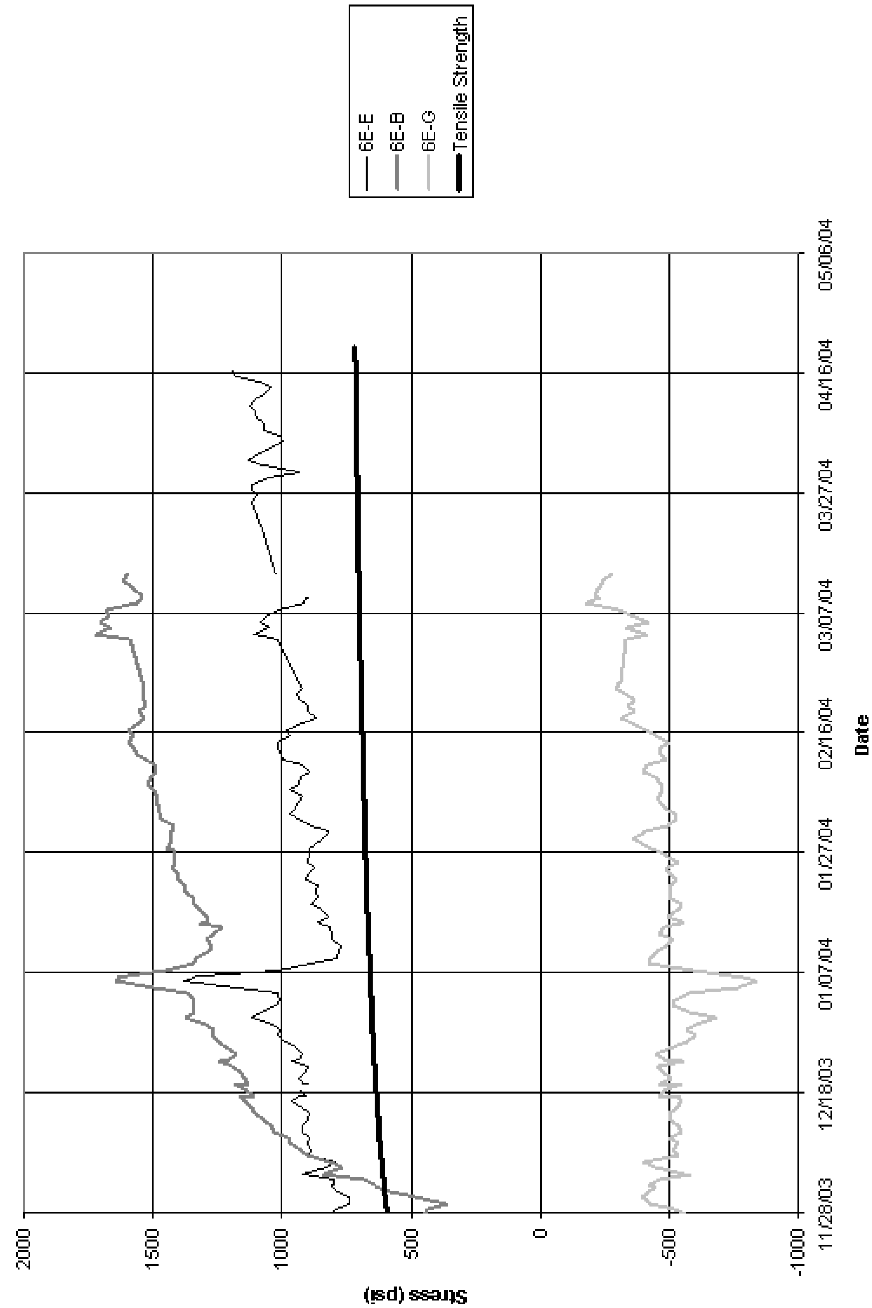

Figure D-8: Potential Deck Stresses at Gauge Location 6E (Interval 2).

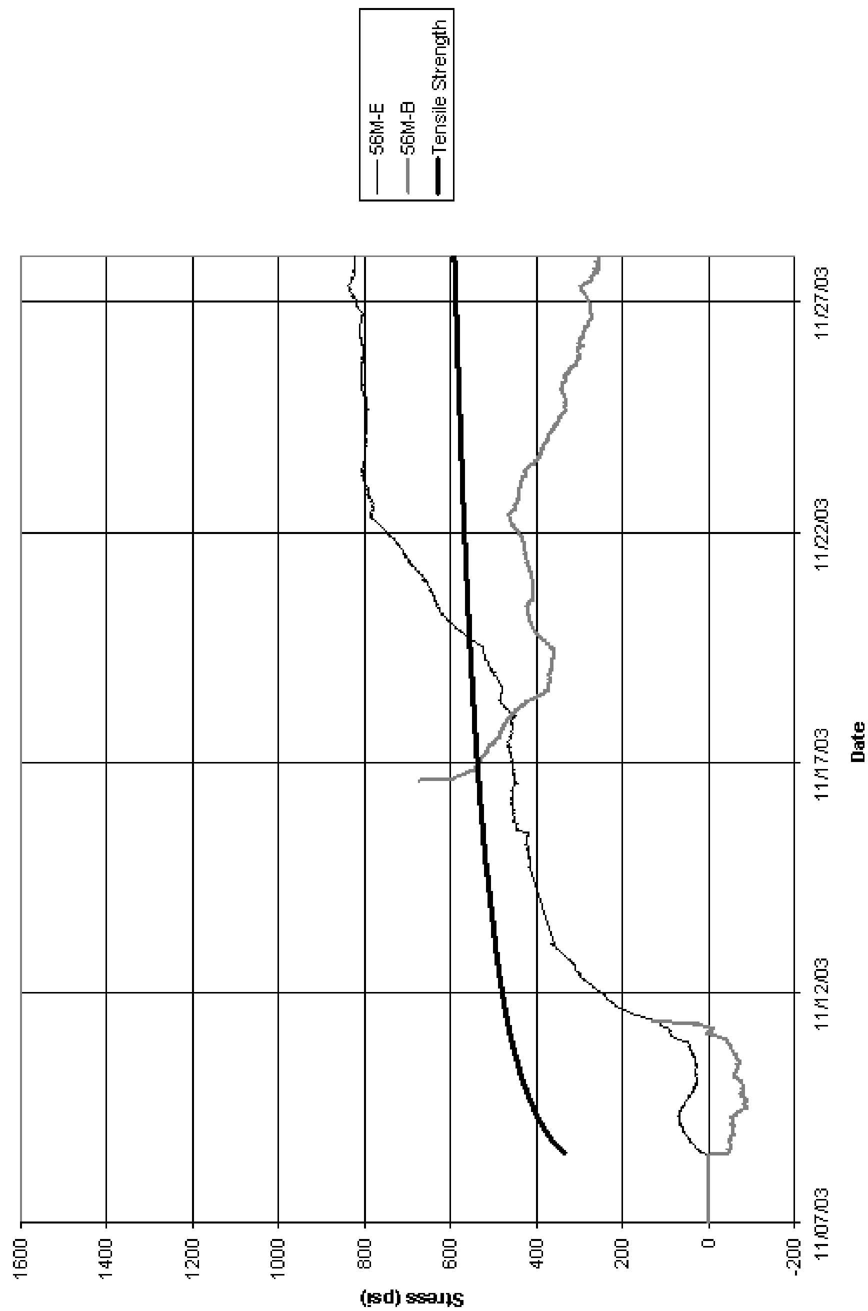

Figure D-9: Potential Deck Stresses at Gauge Location 56M (Interval 1).

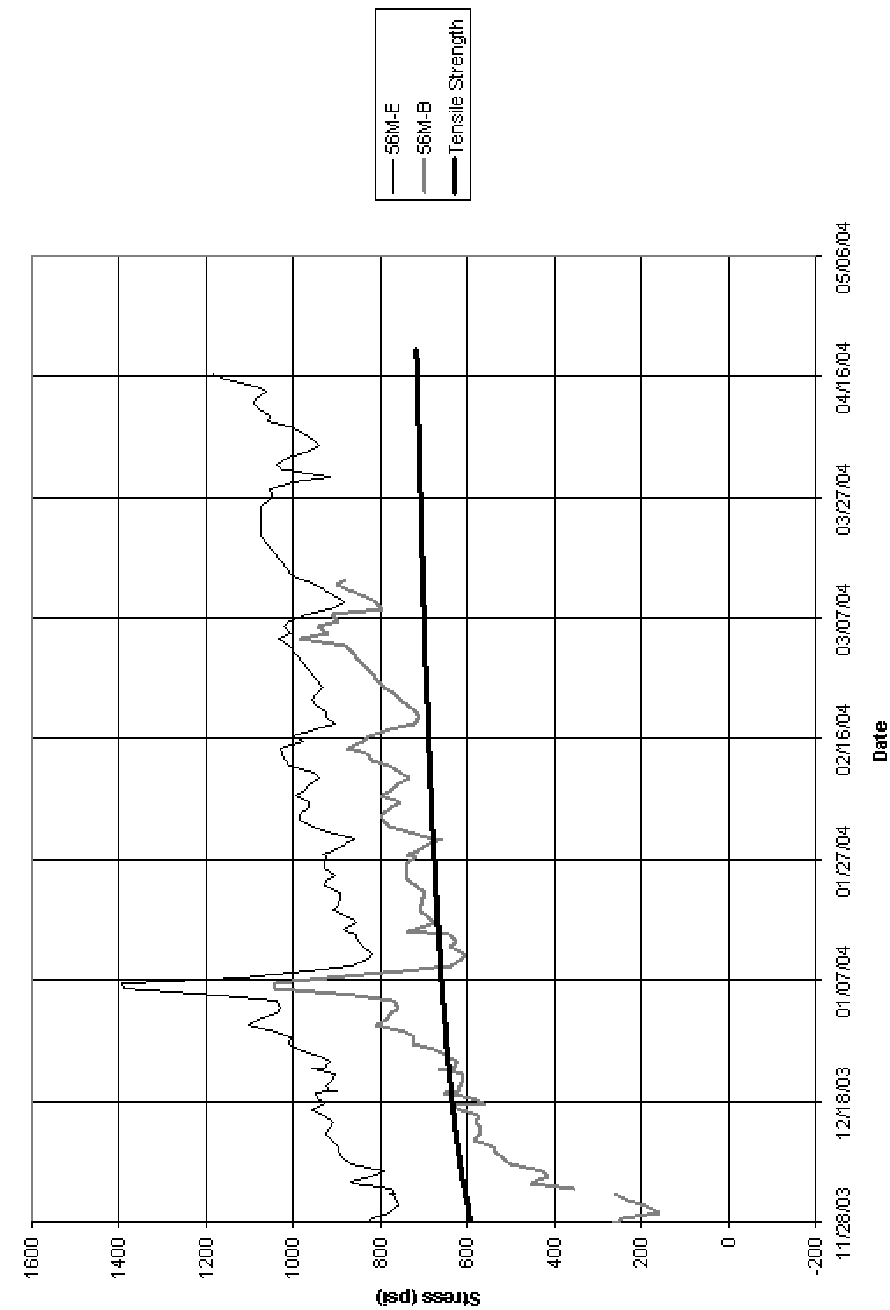

Figure D-10: Potential Deck Stresses Gauge Location 56M (Interval 2).

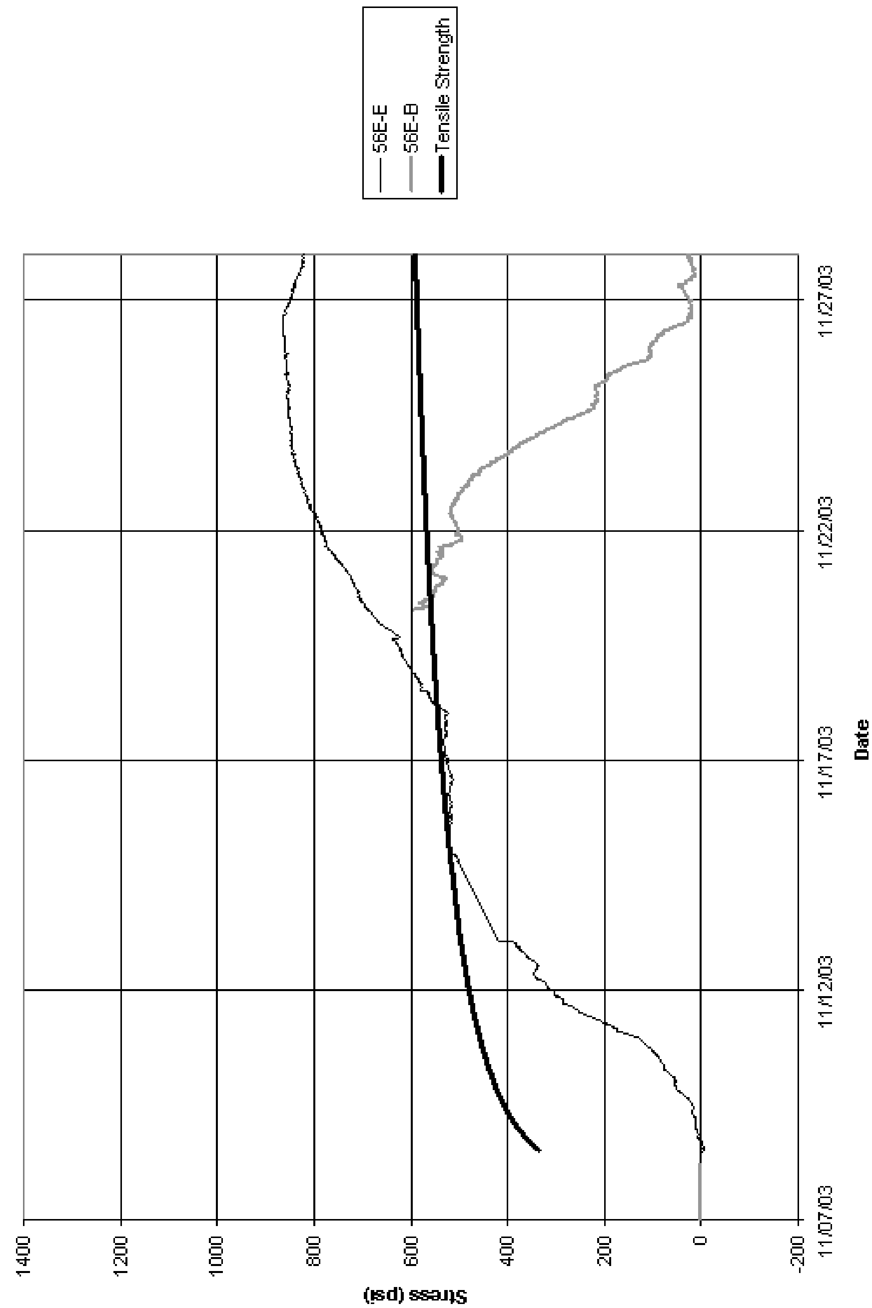

Figure D-11: Potential Deck Stresses at Gauge Location 56E (Interval 1).

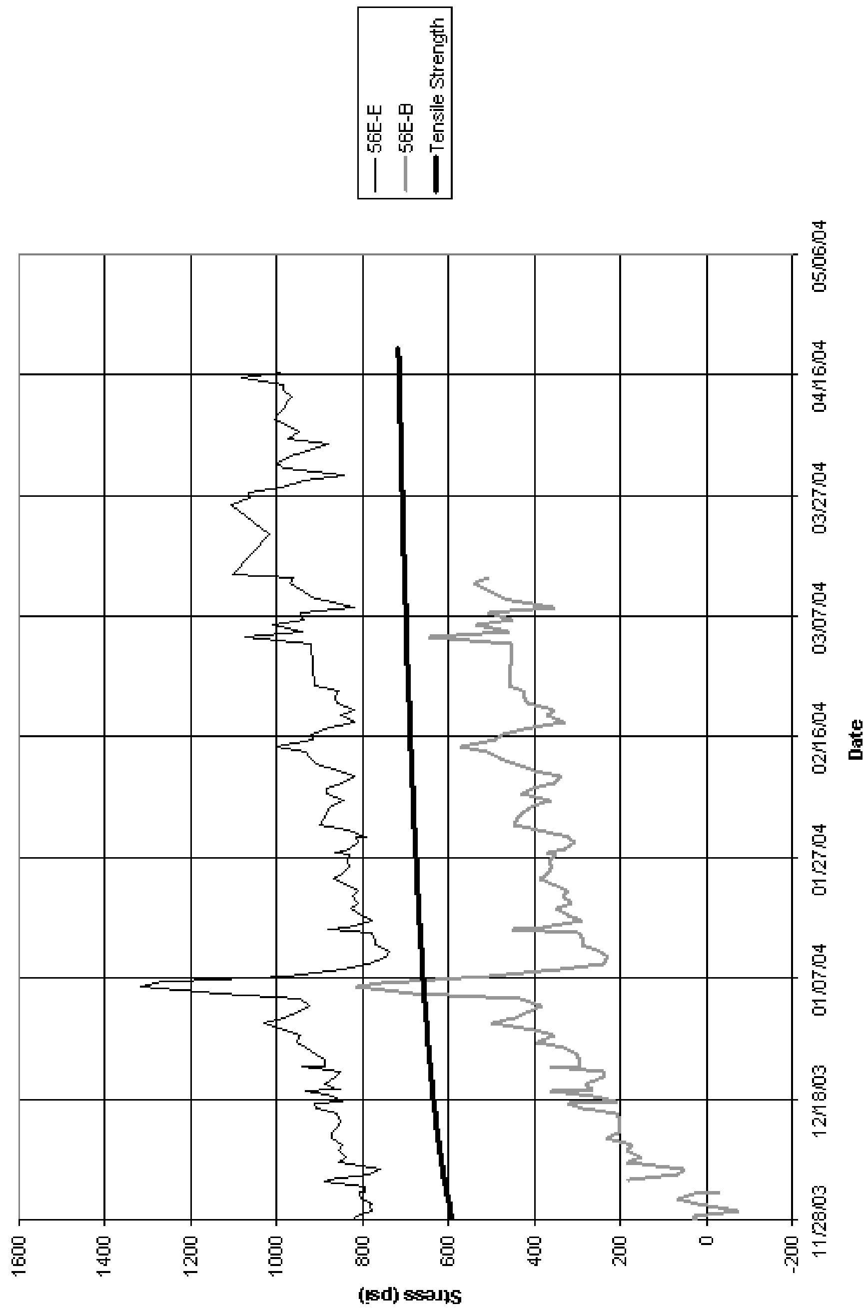

Figure D-12: Potential Deck Stresses at Gauge Location 56E (Interval 2).