Transverse Cracking of High Performance Concrete Bridge Decks After One Season or Six to Eight Months

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Cracking is a major problem with newly placed concrete decks. These decks tend to develop full depth, transverse cracks and partial depth longitudinal cracks within a few months of the concrete being placed. A literature review showed that several other states had experienced similar problems. A review of data from Ohio bridge decks showed weak correlations between deck cracking and slump, time of year when the deck was placed, shrinkage, chloride permeability and compressive strength, but there was no clear relationship between cracking and any of these properties. Data also suggested that using a coarse aggregate with an absorption > 1% may help mitigate deck cracking but will not always stop it.

As part of this study, 3 bridge decks were instrumented. One was a standard class “S” concrete deck and the other two were high performance concrete. The class “S” deck showed only hairline cracking after 1 year, but transverse cracking occurred in the HPC decks. Instruments were placed in the decks to monitor strains. From the data, it appears that cracking is caused by several factors. High heat of hydration caused the plastic concrete to expand. When the concrete sets and cools, tensile stresses develop. Further tensile stresses develop through drying shrinkage. Restraining the deck against normal thermal movement contributes to additional tensile stress. Autogeneous shrinkage, where high heats of hydration cause water evaporation during hydration, and plastic shrinkage may cause more tensile stress.

Recommendations for mitigating cracking include using lower cement contents, adding pozzolans and retarders, using slightly higher water/cement ratios, using larger aggregates, taking steps to limit shrinkage and eliminating restraints.
DISCLAIMER

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ABSTRACT

Cracking is a major problem with newly placed concrete decks. These decks tend to develop full depth, transverse cracks and partial depth longitudinal cracks within a few months of the concrete being placed. A literature review showed that several other states had experienced similar problems. A review of data from Ohio bridge decks showed weak correlations between deck cracking and slump, time of year when the deck was placed, shrinkage, chloride permeability and compressive strength, but there was no clear relationship between cracking and any of these properties. Data also suggested that using a coarse aggregate with an absorption > 1% may help mitigate deck cracking but will not always stop it.

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CHAPTER 1

INTRODUCTION

High performance concrete (HPC) was introduced to the construction market in the 1980s. Through the use of high range water reducing admixtures and pozzolans, chiefly silica fume, fly ash and ground granulated blast furnace slag (GGBS), it was possible to produce high strength concrete with very low permeability.

Bridge decks seemed like an ideal application for HPC. The major problem with bridge decks is corrosion of the steel due to ingress of chloride ions from road salt. With its low permeability, HPC would provide superior resistance to chloride ion penetration. The high strength would make the concrete less likely to crack.

Unfortunately, the HPC did not quite live up to expectations. HPC bridge decks have shown a tendency to crack. The predominant cracks are equally spaced, full depth, transverse cracks which are wide enough to allow water, and therefore chloride ion, penetration. Longitudinal cracking, often over reinforcing bars, has been seen. In some cases, cracking occurs almost as soon as the deck is put in service, but in most cases the cracking occurs after 6-8 months.

The problem of deck cracking is perplexing as there does not seem to be a single cause or an easily identifiable set of causes. There are reported cases of similar bridges which were constructed at the same time with the same materials by the same contractor having extremely different outcomes. One bridge will exhibit severe cracking while another deck will not crack at all. The problem is not confined to a single type of bridge as cracked decks are found on long and short bridges, single and multi-span bridges, straight and skewed
bridges and steel stringer and precast concrete stringer bridges. The problem occurs throughout the state and is not confined to a single area.

This study looks at possible causes of deck cracking. This is done by examining recent studies on the problem, looking at deck cracking data from the Ohio Department of Transportation and by instrumenting 3 bridge decks. Conclusions about the causes of deck cracking and recommendations on preventative measures are also presented.
CHAPTER 2
LITERATURE REVIEW AND REVIEW OF EXISTING DATA

2.1 INTRODUCTION

High performance concrete (HPC) has been used for bridge decks since the early 1990s. Because HPC tends to be denser than normal concrete, engineers believed that HPC would improve deck life by preventing chlorides from attacking steel rebar. It was also believed that the higher tensile strength and lower shrinkage of many HPCs would make the decks more resistant to cracking.

It has been observed that many bridge decks constructed using HPC develop longitudinal and transverse cracks at various stages of the bridge life. These cracks are often more numerous than those found in normal concrete decks. However, since both normal and high performance concrete decks crack, many studies have been conducted on the subject of deck cracking. Because cracking of HPC decks is contrary to what was expected, additional studies have been conducted on properties of HPC as they relate to deck cracking.

2.2 LITERATURE REVIEW

Griffin, Khan and Lopez (2004) looked at deck contraction induced deflection in a high performance concrete bridge. The purpose of the study was to calculate the deflections of a high performance concrete (HPC) highway bridge due to shrinkage and temperature changes in the deck and to compare with the analytical results obtained from the finite element model of the bridge. The bridge under investigation is a four span structure using precast prestressed girders with composite deck over Interstate 75 in
Henry County, Georgia. The bridge was constructed in two phases and evaluation of shrinkage and temperature induced deflections was limited to Phase 2. Vibrating strain gages were installed at key places. The strain measurements in the bridge deck showed significantly larger curvatures and approximately 100 microstrains less compression at the midplane than predicted by the analytical model.

The experimental data suggested that thermal contraction was the primary cause of the additional displacement. Additional displacements were also caused by shrinkage contractions in the deck and the shrinkage deflections were relatively small compared to effects of thermal contractions. The research recommends that the effects of deck contraction to be considered during design. It also recommends additional research on autogenous shrinkage of high performance concrete decks and its effect on deflection.

Babaei (2005) investigated the method to mitigate transverse cracking in concrete bridge decks. The reasons for transverse cracking were observed to be the following:

- **Plastic Settlement Cracking**: This type of cracking is over the topmost bar that is usually transverse in decks with longitudinal steel girders. The type of cracking occurs because concrete attempts to settle, but it is restricted from settlement by the reinforcing bars. The factors that affect settlement cracking are slump, bar size and cover depth.

- **Thermal Shrinkage Cracking**: The peak temperature of curing concrete is usually reached in about 12 to 18 hours after pouring of the deck. At this time concrete starts to harden and cools down and shrinks. The longitudinal steel girders restrain this shrinkage and tensile stresses develop and result in cracking. The difference in deck and beam temperature contributes at an approximate rate
of 5.5 microstrain/degree F. This is the average coefficient of thermal shrinkage of concrete.

The study suggests that the thermal shrinkage of concrete can be controlled by controlling the differential deck/beam temperature during cure. If the temperature differential between the deck and the beam exceeds 40°F within the first 24 hours, it is most likely that there is cracking of the deck during the curing period. Especially during cold weather cure, the temperature differential can reach up to 90°F or more causing cracking during the curing period. The recommendations to minimize deck/beam differential temperature are

- Use less cement
- Replace cement with fly ash or slag
- Use retarders
- Do not over-insulate the deck in cold weather and enclose and heat beams

**Drying Shrinkage:** The drying shrinkage cracking occurs after curing. The study also identifies the factors affecting drying shrinkage as aggregate mineralogy, cement source and type, admixtures, excessive water in the mix and excessive cement paste in the mix. The research recommends the use of hard aggregate with lower absorption, use of low heat type 2 cement and to minimize the use of admixtures. It also suggests that less water in the mix leads to reduction in drying shrinkage. This can be made sure by using a lower slump concrete, larger aggregate, less paste, water reducers and concrete at low temperatures. Drying shrinkage strains are superimposed on the thermal contraction strains.
Thus it is the combination of shrinkage and thermal contractions which crack the decks.

- **Cracking Due to Flexure:** Cracking of this nature is transverse and it occurs over the internal supports. The bridge deck placement sequence also plays an important role in flexural cracking.

Russell (2004), in NCHRP Synthesis 333, provides useful recommendations on the improvement of bridge deck performance. The recommendations can be classified into the following categories

- **Concrete Materials:** The following materials and criteria prove beneficial to enhance the bridge deck performance.

  1) Use of type 1, 2 and IP cement
  2) Use of fly ash and silica fumes (these tend to lower peak heat of hydration)
  3) Aggregates of low modulus of elasticity, low coefficient of thermal expansion and high thermal conductivity
  4) Largest possible aggregate size
  5) Use of water reducing admixtures
  6) Water/cement ratio of around 0.4
  7) Concrete permeability per AASHTO T277 in range of 1500 to 2500 coloumbs.

- **Reinforcement:**

  1) The use of epoxy coated reinforcement
  2) Minimum trasverse bar size and spacing
• **Construction and Design Practices**

1) Minimum concrete cover of 64mm (2.5 in) should be provided.

2) Use a latex modified or dense concrete overlay

3) Apply wet curing for at least 7 days

4) Minimize surface evaporation by using fogging equipment.

5) Apply curing compound after wet curing to enhance the concrete properties and to slow down shrinkage.

Linford and Reaveley (2004) provided recommendations to minimize transverse cracking of bridge decks. The study included the investigation of transverse cracking of decks of bridges as part of reconstruction of I-15 in Utah. It was found that the deck cracking was influenced by the size of concrete placement. The research also discussed the effect of restraint on deck cracking. Usually, rigid attachment between the concrete deck and the steel girders is essential due to establish composite action. This rigid attachment leads to transverse and diagonal crack as the concrete shrinks. The study also suggests non uniform placement of the shear connectors coupled with segmental placing of the concrete deck to reduce cracking.

Frosch, Blackman and Radabaugh (2002) investigated the bridge deck cracking in various superstructure systems with specific reference to bridges in Indiana. The study suggests that the primary cause of cracking is the restraint provided by the steel girder to the expansion and contraction of concrete. Since the reduction of restraint is not possible due to economic advantages of composite construction, the following recommendations have been made.

- A minimum of 7 day curing period is recommended to reduce drying shrinkage.
Proper aggregate selection and gradation, mix design help reduce shrinkage.

Use lower strength concrete as the use of high strength concretes increases transverse cracking in decks.

Additional reinforcement should be provided to prevent yielding of reinforcement that will result in uncontrolled cracking.

The National Center for Transportation and Industrial Productivity (2002) issued a circular on deck cracking in New Jersey. This study concluded that the following factors contribute to deck cracking:

- A large relative stiffness of the girders with respect to the deck.
- Bridges have a high end fixity
- Use of thin decks

The study did not find a relationship with skew or bearing type. The study recommends:

- Reduce cement content to 650-660 pounds/yd³.
- Wet cure at least 7 days.
- Place areas of positive moment first.
- Place a limit on strength.
- Do not try to satisfy deflection requirements by a large margin to avoid using relatively stiff girders with less stiff decks.

Xi, Abu-Hejleh, Suwito, Aziz, Xie and Ababneh (2003) have assessed the cracking problems in early age concrete. The study identifies the different types and suggests recommendations to minimize early age cracking.

- Autogeneous shrinkage- Autogenous shrinkage is caused by the consumption of water during the hydration process of cement particles and it is accompanied by a
reduction of relative humidity in the concrete and an increase in the surface tension in capillary water. Autogeneous shrinkage is not due to the exchange of moisture between concrete and the environment. Autogeneous shrinkage occurs even if there is no moisture exchange at all. Concrete mix designs with a low water to cement ratio (.33 - .40) exhibit high autogeneous shrinkage. To reduce early age cracking, higher w/c ratios are needed.

- Drying Shrinkage: Hardened concrete loses water when exposed to a relative humidity less than 100%. The shrinkage strain for commonly used concrete ranges from 500 to 1000 micro strain. The two important factors in drying shrinkage are the ultimate shrinkage strain and the rate of shrinkage of the concrete. For a higher shrinkage rate, concrete shrinks higher in a given period of time and this leads to deck cracking. For early age concrete, creep partially relieves the shrinkage stress and hence reduces the possibility of deck cracking. The range of parameters that influence drying shrinkage include aggregate type, water content, cement content, curing methods and placement temperature. Among various constituent phases, cement paste is the primary phase that causes drying shrinkage and hence lowering the cement content decreases drying shrinkage. However it is difficult to decrease the rate of shrinkage.

- Plastic Shrinkage: This is caused by a rapid loss of moisture on the concrete surface while it is still in a plastic state. It occurs when the rate of evaporation exceeds the rate of concrete bleeding. During the plastic period, mixing water and cement particles form a suspension system. The stability of this system depends on the distance between cement particles. The dried zones in the fresh concrete
will develop in regions of large porosity. The coexistence of the dried zone and the saturated zones result in tensile strength of concrete. When tensile stress exceeds the tensile capacity of concrete, the cracks develop in concrete. The study suggests proper curing methods such as fog mist or curing applied to the concrete surface to reduce the evaporation rate. This helps reduce plastic shrinkage.

The study also recommends the use of higher reinforcement ratio to restrict cracking. The study also limits cement content to a maximum of 470 lb/yd³. The study recommends a water/cement ratio of around 0.4. It also recommends construction practices like casting decks when air temperature ranges from 45° F to 80° F. Also suggested is that concrete placement do be avoided when the evaporation rate is above 0.2lb/ft² hr and to apply immediate fogging to all concrete decks until the surface has been covered by final cure.

Two other bridge deck cracking reports (Brown et al, 2001, and Hadidi et al. 2003) reached similar conclusions to those cited above.

Huo and Wong (2000) conducted a study on the early age properties of HPC. They determined that even though total shrinkage of HPC is less than that of conventional concrete, it has higher early-age shrinkage. In a restrained system, this early shrinkage may lead to early-age cracking as the concrete will develop high tensile stresses before the full tensile strength is developed. The paper presents analytical and experimental results of the behavior of several concrete deck specimens with alternative curing methods. Two models are used to predict the stresses in the concrete for four different techniques of curing. One model uses the empirical ACI equation for shrinkage strain and the other is derived from theory of elasticity and suggests appropriate curing methods that can minimize moisture loss and hence prevent transverse cracking. This
paper suggests recommendations for proper curing techniques to minimize transverse cracking.

One area of shrinkage which has been suggested as a cause of shrinkage in HPC is autogeneous shrinkage. Autogeneous shrinkage occurs during the first day or two after the concrete has taken its final set. When a concrete has a high cement content, a large heat of hydration will be generated. This heat may be enough to cause the water in the concrete to boil out. If the water/cementitious material ratio is low, the concrete may begin to lose structural water and shrink.

Hossain, Pease, and Weiss (2003) conducted research on quantifying early-age stress development and cracking in low w/c concrete using the restrained ring test with acoustic emission. According to the research, early age cracking occurs when volumetric changes associated with shrinkage and temperature reduction are prevented. Early age cracking is influenced by strength gain, stiffness development, creep, shrinkage and degree of restraint and toughness. The study recommends the ring test to be used as a comparative test to screen potential mixture designs. AASHTO developed a provisional standard ring test that establishes specimen geometry. Although the ring test does not provide a direct method for quantifying the development of stress, this research has developed quantitative stress solution based on the results of the ring test. The method also takes into account the stress relaxation in the material. Acoustic-emission testing was performed to evaluate micro cracking and crack propagation. The results indicated that as the restraint increased, micro cracking increased as a part of the stress relaxation process.

Lee, Lee and Kim (2003) studied the stresses created due to autogeneous shrinkage of high performance concrete (HPC). Finite Element method was used to
determine the stresses. A model was proposed to predict the autogeneous shrinkage of concrete. It was found that the autogeneous shrinkage of concrete developed at a much higher rate than the tensile strength of concrete. The paper also evaluated the effect of water/cement ratio and fly ash percentage on autogenous shrinkage of high performance concrete.

Dias (2003) studied the effect of mix constituents, retarding and air entraining admixtures and additives, and environmental factors on plastic shrinkage of concrete. The influence of mix constituents, retarding and air entraining admixtures and the effect of environmental factors of plastic shrinkage was established.

Branch, Hannant, and Mulharon (2002) conducted research on the factors affecting plastic shrinkage for high performance concrete. Test procedures included the measurement of tensile stress-strain performance during the first five hours after mixing, negative pore pressure development and free shrinkage during the first twenty four hours. Test specimens included concrete of various high strength mixes and supplementary cementitious material such as microsilica, pulverized fuel ash, granulated slag and metakaolin. Plastic shrinkage was calculated using restrained ring tests with samples sealed and samples exposed to wind. It was observed that plastic shrinkage was higher when microsilica and wind were present.

Cope and Ramey (2003) suggested ways to reduce shrinkage to minimize cracking. Shrinkage compensating cement is recommended. This cement contains shrinkage reducing admixtures (SRA) that reduce early drying shrinkage of bridge decks.

Heat of hydration generates thermal stresses which results in concrete cracking. High heat of hydration results in setting of concrete at a higher temperature. Then the
concrete cools down from the high temperature at the point of hydration to surrounding temperature. This results in high tensile strains generated due to the presence of restraint. High heat of hydration can be generated due to the following causes:

- The use of high early strength cement.
- High environmental temperature which accelerates hydration.
- The use of accelerators.
- High fineness of cement
- High cement content in the mix

The literature review shows that a large number of studies have been done on deck cracking. While these studies may have varied recommendations, the main conclusions are remarkably similar. In summary, cracking of bridge decks is largely attributed to:

- Contraction of the decks due to thermal effects. Concrete decks heat up due to heat of hydration and set while hot. Contraction due to cooling causes tensile stress.
- Shrinkage. Decks crack due to plastic shrinkage, autogeneous shrinkage and drying shrinkage.
- Overly restrained deck systems.

2.3 REVIEW OF EXISTING ODOT DATA

When ODOT began using HPC decks, contractors were required to collect data on the decks. Data regarding the deck’s geographical location, date of casting, type of mix, ninety day chloride penetration value, average 90 day shrinkage, maximum temperature differential, air percentage, slump and compressive strength were collected. At the beginning of this study, the current deck condition for about 300 of these decks
was obtained from ODOT. All data was not available for all decks, so the study was confined to those decks for which all details were known (115 decks). The aim of collecting and analyzing this data was to ascertain any direct relationship of deck cracking to any of the factors mentioned in the literature survey.

The decks were inspected by ODOT inspectors for cracking and the results are presented in this section. For the purpose of categorization, the cracking severity was grouped into three categories:

- **Fair**: Those decks which had some noticeable cracking
- **Good**: With minor cracks
- **Best**: Those decks which showed no cracking

The number of bridges in each category was found and then the data was plotted in terms of the number of decks in each category. Because not all data was available for all deck, the numbers do not always add up to 115 decks.

**Effect of Slump**

For all decks, the slump generally varied from 4 to 9 inches (Figure 2.1). While decks with higher slumps tended to perform better, there is no strong correlation to slump. Decks with higher slumps may have more water and/or higher air contents, although there are a large number of factors which may also affect slump. More water may limit autogeneous shrinkage and more air would increase freeze/thaw durability. However, more water would increase drying shrinkage and more air would weaken the concrete. Thus, it is not possible to draw a strong conclusion from this data.
Effect of Mix

Ohio DOT has four different HPC mixes specified for bridges. Table 2.1 shows the constitutive materials. The mixes used for most of the decks are either 3 or 4 as those were intended for deck usage. Only one deck has been cast with mix number 2 and it did not develop any cracks. Decks cast with mix number 4 show better results than those cast with mix number 3.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Design Values</th>
<th>Mix quantities (cu yd lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>w/c max</td>
<td>Slump</td>
</tr>
<tr>
<td>1</td>
<td>0.38</td>
<td>8</td>
</tr>
<tr>
<td>2</td>
<td>0.38</td>
<td>8</td>
</tr>
<tr>
<td>3</td>
<td>0.4</td>
<td>8</td>
</tr>
<tr>
<td>4</td>
<td>0.4</td>
<td>8</td>
</tr>
</tbody>
</table>

**Table 2.1: ODOT HPC Mix Designs**
Effect of Penetration Values

The ninety day rapid chloride penetration values for the decks are in the range from 300 to 1800 coulombs with just one deck having a rapid chloride penetration value lying in the range of 3300 to 3600. In general, decks with low permeability perform better, but the relationship is not strong. Some high permeability decks perform well and some low permeability decks crack.
Effect of Average Shrinkage

As seen in Figure 2.4, decks with lower shrinkage values tend to perform better, but the correlation is not strong.

**Figure 2.4 Effect of Shrinkage on Deck Cracking**

Effect of Compressive Strength

The results of the effect of compressive strength are counter-intuitive. Concrete with higher compressive strengths should have higher tensile strengths and should, therefore, be more resistant to cracking. However, the data do not support this. The best performance seems to be for concrete with compressive strengths of 6-7 ksi. Higher strength concrete would tend to have lower water contents and higher heat of hydrations, which would increase both the autogeneous shrinkage and the differential temperature at the time of casting. Both contribute to increased deck cracking. The 6-7 ksi concrete is
probably a good compromise between strength, heat of hydration and shrinkage characteristics.

![Figure 2.5 – Effect of Compressive Strength on Deck Cracking](image-url)

**Effect of the month of the year**

The data show that decks are cast from February to December. Cracking has been observed in the decks regardless of the month it was cast. The best performing decks have been cast in the months of July and August but the data show a large amount of scatter. For example, the month of August has the highest number of uncracked decks and almost an equal number of fair decks. While it can be generally said that decks cast in comparatively warmer months performed better, a conclusion cannot be based on this as the actual weather condition at time of casting is not known. To the extent that this is true, it may be due to the differential temperature effects. As previously noted, the deck sets while the concrete is still hot due to heat of hydration. During cold weather, the
amount of cooling will be greater leading to greater tensile stresses during the time the concrete has the lowest tensile strength.

![Bar graph showing the effect of month of casting on deck cracking.](image)

**Fig 2.6 – Effect of Month of Casting on Deck Cracking**

**Effect of Geographic Location**

Figure 2.7 seems to show trend of cracking concentrated in some parts of the state. Details available for Districts 12 and 3 report no cracking or very minor cracking. The decks in District 8 show a higher incidence of cracking. However, inspection of the decks by the research team revealed little difference in cracking between Districts 12 and 8. It appears the deck rating is somewhat subjective, and the difference appears to be in how different districts rate decks.
Summary of ODOT Data

The study of the relationship between the various factors affecting transverse cracking of concrete decks and the actual cracking reveals that there is no definite correlation between the factors causing cracking and the actual cracking at the site.

2.4 DATA ANALYSIS OF ABSORPTIVITY DATA

Engineers in District 12 noted that decks using highly absorptive coarse aggregate (absorption > 1%) did not crack. Using data on absorption from throughout the state, the effect of absorption can be seen in the following graphs.
Fig 2.8 Effect of Absorption on Deck Cracking

From the above graphs it can be established that decks which did not crack had absorptivity above 1.0. However, simply having absorption >1.0 does not guarantee the deck won’t crack. Thus, having a higher absorption aggregate appear to help, but is not sufficient as a single factor. The mechanism of why higher absorption aggregate is beneficial is not clear, but it may relate to autogeneous shrinkage. The aggregate absorbs
water and it is possible that this water is then made available during the hydration process, mitigating the drying effect of the heat of hydration.

2.5 SUMMARY OF THE LITERATURE SEARCH AND ANALYSIS OF ODOT DATA

Based on a search of the literature, it appears that cracking of bridge decks is attributed to:

- Contraction of the decks due to thermal effects during the hydration process. The heat of hydration heats both the plastic concrete and tops of the girders, causing an expansion before the concrete has set. After the concrete sets, the system cools and contracts, resulting in tensile strain development in the system.

- Shrinkage. Decks crack due to plastic shrinkage, autogeneous shrinkage and drying shrinkage. Data from ODOT bridges did show a weak correlation with shrinkage values. The reason the correlation was not stronger is probably that the shrinkage values were obtained from the AASHTO T160 test. This test will not measure autogeneous or drying shrinkage which contribute to deck cracking. This test is performed under standard conditions of 73°F and 50% relative humidity. Concrete decks are subjected to varying levels of relative humidity which affects to the amount of shrinkage. Shrinkage is also affected by the surface area to volume ratio, which is very different for the deck as compared to the standard T160 specimen. Thus, the T160 test gives an indication of shrinkage potential and allows for a comparison of the relative shrinkage between two concrete mixes, but it does not measure the actual shrinkage which would occur in the deck.
• Overly restrained deck systems.

Analysis of data from HPC deck showed that:

• There is a weak correlation between deck cracking and shrinkage.

• Deck using a mix with blast furnace slag performed slightly better than those without.

• Decks with compressive strengths of 6-7 ksi performed the best. This was attributed to the concrete having enough strength to resist cracking, but a low enough cement content that effects of excessive heat of hydration were minimized.

• Absorption of the coarse aggregate appears to be important. Decks which utilized concrete with coarse aggregate absorptions less than 1% always cracked. Decks which did not crack had coarse aggregate absorptions above 1%

• There was no correlation found between deck cracking and time of the year the deck was cast, the rapid chloride penetration, the geographic location or slump.
CHAPTER 3

BRIDGE DATA AND DECK INSTRUMENTATION

3.1 INTRODUCTION

This project seeks to determine the cause of early age deck cracking. It has been found that many bridge decks in Ohio crack within the first year of life. The cracks are usually transverse to the beams and are usually approximately equal in spacing. Longitudinal cracking is also observed.

As stated in Chapter 2, there was no obvious correlation between any property and cracking, except for absorption of the coarse aggregate. However, even that was not an absolute correlation. To try to determine the cause of deck cracking, three bridge decks were instrumented and monitored.

One deck was DAR-127-1977 in Darke County. The other two bridges were left and right bridges on Interstate 70 in Clark County (CLA-70-0699). The Darke County Bridge (DAR -127) used standard Class “S” concrete. The twin, but separate, Clark County structures (CLA-70) used ODOT HPC mix HP3. All three decks were composite, phased construction. A closure pour was used to connect the two phases.

The decks in Clark and Darke County were monitored to obtain the strain history of various positions in the deck along the span. Strains and corresponding temperatures at various points on the deck were measured at various intervals. After, the deck was cast, the data was taken every 30 minutes. This time interval was lengthened so that after about 1 month, the data was taken every 3 hours.
3.2 INSTRUMENTATION OF THE DECK

In order to investigate the effects of temperature and shrinkage on the bridge deck, strains were to be measured at significant locations in the bridge. Each bridge was constructed in 2 phases. For each phase vibrating wire strain gages were placed at four locations:

- At the midspan of an end span, between two girder lines.
- At a pier line, between two girder lines.
- Over a beam at the midspan of an end span.
- Over a beam at a pier line.

At each location, a gage was placed just below the top mat of rebar and just below the bottom mat of rebar. Thus, 8 gages were placed in each phase for a total of 16 gages per deck. The locations of the vibrating strain gages were chosen in a manner that represents the location of minimum and maximum restraint the steel girder offers to the concrete deck on top. The steel girders provide maximum restraint near the beam-pier intersection and minimum restraint at the midspan.
Fig 3.1 Clark County Bridge – Location of Vibrating Wire Strain Gages.
Fig 3.2 Darke County Bridge - Location of Vibrating Wire Strain Gages
The vibrating wire strain gages were model VCE 4200 from Geokon. These gages are designed to be embedded in concrete. When embedded, the wire grips (Figure 3.3) move along with deformations of the surrounding concrete. These deformations alter the tension in the wire and the natural frequency of vibration is also altered. The “pluck and read” coils are small electro-magnets. When connected to a data system (a Campbell Scientific CR10X in this case), the data system energizes the “pluck coil”. The magnetic pulse “plucks” the wire and it begins to vibrate at its natural frequency. The vibrating wire cuts the magnetic field of the read coil, sending pulses back to the data system. The data logger measures the frequency of the pulses and this is used to evaluate the strain.

**Fig 3.3 Vibrating Strain Gage (Courtesy of GEOKON Inc.)**

The vibrating wire gage has an advantage over the conventional electric or semiconductor gages as it measures frequency of vibration of the wire, which doesn’t vary with respect to cable resistance or contact resistance. As will be discussed in Chapter 4, the gage is also able to correct for temperature changes. A thermistor attached to the gage measures temperature.
The gages were tied to the rebar at the required locations using a tie wire as shown in Fig 3.4. Foam spacers were used to separate the gage from the rebar. Note that one gage (Top gage) is tied just below the top mat of the rebar. This places the gage just above the mid-height of the slab. The bottom gage is tied below the bottom mat of rebar, placing it either in the cover concrete or in the haunch concrete.

After the concrete was placed, the strain gages were read every hour. After a month, the gages were read every 3 hours. The research team had to travel to the bridge to retrieve the data, so reading every 3 hours allowed the data loggers to run for 1-2 months without losing data. However, in some cases the team was unable to get to the bridge(s) in time to retrieve data, so some data was overwritten and lost. Thus, graphs of the data will have some gaps. However, these gaps do not affect the overall integrity of the data.

**Fig 3.4 Vibrating strain gages tied to the re-bar.**
3.3 SHRINKAGE OF CONCRETE

During each of the six phases of construction (2 on Darke County and 4 on Clark County), shrinkage samples meeting AASHTO M210-01 (ASTM C490-97) were cast. The sample size was 4” x 4” x 11.25”. The samples were wet cured for 7 days to duplicate the curing of the deck. The shrinkage test was conducted according to AASHTO T 160-97 and ASTM C 157-23.

The specimens from DAR-127-1977 were tested as required by AASHTO T160 using a comparometer. However, this is a mechanical device and no strain can be measured until the specimens are demolded. Also, there is some “play” in the mechanism which the research team thought created some inaccuracy in the measurements. For CLA-70-0699, vibrating wire gages were placed in the specimens. This allowed continuous measurement of the strain with an accuracy of 1 microstrain.
CHAPTER 4
RESULTS AND DATA ANALYSIS

4.1 CRACKING IN THE DECKS

During the monitoring phase, the decks of the Darke County bridge (DAR-127-1977) and the two Clark County bridges (CLA-70-0699) were inspected for cracking. One year after Phase II was cast, the Darke County bridge showed only minor, hairline cracking. Approximately 8 months after Phase II was cast in the Clark County bridges, cracking was found in Phase II. The cracks appeared to be full depth and were spaced at 6-8 ft. intervals.

4.2 RESULTS OF MONITORING THE BRIDGE DECKS

4.2.1 Concrete Hydration

The chemical reaction of hydration produces heat over the curing period of the concrete. The temperature distribution of the concrete structure, during the period of hydration of concrete, depends upon the following factors.

- The heat of hydration: The magnitude of the energy released during the process of hydration determines the heat gained by the concrete structure. This, in turn, impacts the temperature distribution of the structure. Heat of hydration is a function of cement content, the physical and chemical properties of the cement and the use of admixtures.

- The heat of conduction: The rate at which heat is transferred within the body of concrete structure affects the temperature distribution of the system. The heat of conduction depends on the thermal conductivity of concrete and the supporting materials, specific heat and density of the material.
- **The heat of radiation:** The rate at which heat is lost to surroundings affects also affects the temperature distribution during the process of hydration. The higher the rate of conduction and radiation, the faster is the heat lost by the concrete structure. A significant increase in temperature may occur, especially in thick members when the dissipation of heat by conduction and radiation is smaller compared to the heat of hydration liberated during the process of hydration. Because the conductivity of concrete is relatively low, steep temperature differential can be created between the interior mass of concrete and the surfaces. Radiation will also be limited by insulating materials. Formwork can act as insulation. In cold weather, additional insulation may be intentionally added to prevent heat radiation in order to prevent freezing.

As noted in Chapter 3, thermistors attached to the vibrating wire gages measured the slab temperature. This temperature was recorded at 30 minutes for the initial pouring and hydration phase for each phase. An increase in temperature of about 15 to 30°C (27 to 54°F) is observed during the first 18 to 25 hours. Then the deck cools down during the subsequent 200 hours. This heat gain will be shown to have a significant influence on deck behavior.

**4.2.2 Reference Strains**

When measuring strain with vibrating wire strain gages (VWG), it is necessary to have a reference, or zero, reading, \( R_0 \). The actual strain is then found from \( R_1 - R_0 \), where \( R_1 \) is the reading of the strain gage. For concrete, determination of this reference strain is often difficult. The VWG is embedded in the concrete while it is plastic. Clearly any deformation of the plastic concrete is meaningless. It is necessary to take the zero reading at the time the concrete sets. The reference point for the measurement of strain
is taken at the time the peak value of temperature due to heat of hydration occurs, as shown in the figure 4.1. In theory, this is approximately the time when the concrete takes its final set, where final set is defined as the point where the concrete begins to develop measurable mechanical properties.

There is some justification for this choice. Miller et al. (NCHRP 519) cast concrete slabs on top of continuous, precast type III AASHTO “I” girders and monitored the reactions. During the time where the concrete temperature was rising, there was no change in the support reactions. However, at about the point of peak temperature, the support reactions began to follow the heat of hydration curve. The end reactions decreased as the heat fell while the center reactions increased. Miller et al. attributed this to the setting of the slab. While the heat was rising, the slab had not set. The concrete expanded as it heated, but it had no effect on the structure as the concrete was still plastic. Near the peak of the heat of hydration, the slab set (meaning it began to have measurable mechanical properties). As the heat of hydration fell, the slab cooled and began to contract. This caused a “curling” effect, lifting the ends of the beams and causing the end reactions to fall and center reactions to increase. Thus, the reference strain should be the point where concrete sets and this is usually the point where the heat of hydration reaches its maximum.

The temperature versus time graph is plotted in Figure 4.1 for a gage at the pier line, over a beam and at the bottom of the slab of the Clark County bridge. The reference reading ($R_o$) is taken at this point.
4.3 APPARENT STRAINS

The apparent strains are determined by subtracting the reference strains, as determined at the point of peak heat of hydration, from the measured strains recorded in the vibrating strain gages. Tensile strain is positive.

In this study, a three bridges were instrumented, DAR 127-1977 and CLA 70-0634 R&L. Each bridge deck was cast in two phases, making a total of 6 instrumented phases. The phases are defined as:

DAR-127-1977:

Phase 1 – East half

Phase 2 – West half
CLA-70-0659:

Phase 1 = North half of the right (south) bridge.

Phase 2 = South half of the left (north) bridge

Phase 3 = North half of the left bridge

Phase 4 = South half of the right bridge.

For all three bridges, a closure pour was made to link the two phases together.

The strains as measured by the strain gage at various positions for Phase 1 of the Clark County Bridge and Phase 1 of Darke County Bridge are presented for Figures 4.2 and 4.3 (Refer to Chapter 3 for gage positions)

![Graph showing strains over time for Phase 1 of the Clark County Bridge](image-url)

**Fig 4.2 Clark County Bridge (CLA 70 – 0659) - Phase 1**
Fig 4.3 – Darke Country Bridge (DAR127-1977) – Phase I

The strains measured by the gages placed at the bottom of the slab, over the beam at the pier location (ref. Chapter 3) for the four phases of the Clark County Bridge are presented Fig 4.4.

Fig 4.4 Clark County - CLA-70-0659 – All Phases –Over the Pier, Over a Beam.

Bottom Gage
The strains measured by the gages placed at midspan, at the bottom of the slab between the beam lines for the four phases of CLA 70 are presented in Figure 4.5:

Fig 4.5 Clark County - CLA-70-0659 – All Phases –Midspan, Between Girder Lines, Bottom Gage

The remaining raw strain data is found in Appendix B. The following observations can be made from the strains recorded by the various gages (see Appendix B):

- Many of the gages show tensile strain because the gage contracts more than the concrete. This increases the tension in the vibrating wire of the vibrating strain gage.
- Over the beams, the strains are similar. This is probably due to the restraining effects of the composite connection. The gages between the beams show much higher strains.
- For the two Clark county bridges, the strains recorded in phases 1 and 2 look similar. The strains recorded in phases 3 and 4 look similar. As phases 1 and
2 were the initial pours on each bridge while phases 3 and 4 represent the remaining halves of the bridges.

4.4 CORRECTION OF THE STRAIN READINGS

A VWG consists of a steel wire attached to two end blocks. As the end blocks move, the displacement can be measured by determining the change in the tension of the wire from the change in the natural vibrational frequency of that wire. Because the end blocks are rigidly attached to the structure, strain in the structure can be calculated from the displacement of the end blocks (See Chapter 3).

Now consider a gage which is NOT attached to a structure and which is subjected to a temperature change. The gage will expand or contract with the change in temperature. However, the wire will also expand or contract, so the tension in the wire will not change. Thus, the VWG is self-correcting for temperature. Unfortunately, this self correction breaks down when the gage is embedded in concrete. Steel has a slightly larger coefficient of thermal expansion than does concrete. For a given change in temperature, the VWG will attempt to contract or expand more than the concrete in which it is embedded giving a false strain reading. It is easy to correct for this:

\[ R = (R_1 - R_0) + \Delta T(\alpha_s - \alpha_c) \]

where:
\[ R = \text{corrected strain} \]
\[ R_1 = \text{gage reading} \]
\[ R_0 = \text{zero reading} \]
\[ \Delta T = \text{temperature change} \]
\[ \alpha_s = \text{coefficient of thermal expansion – steel} = 6.7 \times 10^{-6} / ^\circ \text{F} \]
\[ \alpha_c = \text{coefficient of thermal expansion – concrete} = 5.7 \times 10^{-6} / ^\circ \text{F} \]
The values of the coefficients of thermal expansion are those suggested by the gage manufacturer. *Note that the corrected strain reading, R, is the strain due to all effects (load, restraint, creep, shrinkage, etc.) EXCEPT temperature.* Also note that in the absence of load, creep or shrinkage strains, the corrected strain reading, R, is the stress producing strain which would occur if the concrete were restrained against thermal movements. The change in temperature is measured by thermistors on the VWG.

In the calculation above, the strain value, R, does not include the free contraction due to temperature, but this can be easily found:

\[
R_{\text{ext}} = (R_1 - R_0) + \Delta T(\alpha_s - \alpha_c) + \Delta T\alpha_c
\]

which reduces to:

\[
R_{\text{ext}} = (R_1 - R_0) + \Delta T\alpha_s
\]

\(R_{\text{ext}}\) is the total expansion or contraction from ALL effects including temperature. Basically, this what an external strain measuring device would find.

The explanation above is easier to understand with examples. Begin by considering a block of concrete with an embedded VWG. The block is free to expand or contract, thus temperature changes will cause displacements but no stress. If there is a temperature change of -10°F, the gage will read \(R_1 - R_0 = 10\) microstrain (or \(10 \times 10^{-6}\) in/in). This is due to difference in the coefficient of thermal expansion. Applying the corrections:
\[ R = (R_1 - R_0) + \Delta T (\alpha_s - \alpha_c) \]

\[ R = 10 \times 10^{-6} + (-10^\circ F)(6.7 - 5.7)(10^{-6}) = 0 \]

and:

\[ R_{\text{ext}} = (R_1 - R_0) + \Delta T \alpha_s \]

\[ R_{\text{ext}} = 10 \times 10^{-6} + (-10^\circ F)(6.7 \times 10^{-6}) = -57 \times 10^{-6} = \Delta T \alpha_c \]

Since the block is free to contract, there should be no stress producing strain so \( R = 0 \).

The free contraction of the block is given as \(-57 \times 10^{-6}\), which is exactly the answer which would be calculated by the strength of materials solution. An external strain gage would measure \(-57 \times 10^{-6}\).

Now assume the block is completely restrained against any movement. If the temperature decreases \(10^\circ F\), the gage will read 67 microstrain. Calculating the corrections:

\[ R = (R_1 - R_0) + \Delta T (\alpha_s - \alpha_c) \]

\[ R = 67 \times 10^{-6} + (-10^\circ F)(6.7 - 5.7)(10^{-6}) = 57 \times 10^{-6} \]

and:

\[ R_{\text{ext}} = (R_1 - R_0) + \Delta T \alpha_s \]

\[ R_{\text{ext}} = 67 \times 10^{-6} + (-10^\circ F)(6.7 \times 10^{-6}) = 0 \]

Again, the results agree with the strength of materials solution. The concrete block will see a TENSILE, stress producing strain of \(57 \times 10^{-6}\) in/in due to the restraint. However, an external measurement of strain will be 0 because the block is restrained. The remaining 10 microstrain is the “error” due to difference in thermal expansion between concrete and steel.

Finally, assume the UNRESTRAINED block is subjected to a decrease in temperature of \(10^\circ F\) and the VWG measures \(-40 \times 10^{-6}\) in/in. Making the corrections:
\[ R = (R_1 - R_0) + \Delta T(\alpha_s - \alpha_c) \]
\[ R = -40 \times 10^{-6} + (-10^\circ F)(6.7 - 5.7)(10^{-6}) = -50 \times 10^{-6} \]

and:
\[ R_{ext} = (R_1 - R_0) + \Delta T\alpha_s \]
\[ R_{ext} = -40 \times 10^{-6} + (-10^\circ F)(6.7 \times 10^{-6}) = -107 \times 10^{-6} \]

Since the block is unrestrained, R should = 0 if temperature is the only thing effecting the block. Since R is not 0, this indicates that something, other than temperature, is causing an additional 50 microstrain contraction of the block. It may be load, creep or shrinkage. These additional effects will be discussed in a following section.

**4.5 ACTUAL STRAINS**

Actual strains are the strains measured by the strain gages corrected for the differences in coefficients of thermal expansion between the concrete and the gage. The actual strains of concrete at two sample positions in the Clark County bridges are presented in Figs 4.6 and 4.7. Figures 4.8 and 4.9 show sample corrected strains for the Darke county bridge.
Fig 4.6 Raw and Corrected strain – Clark County (CLA-70-0659), Phase I, Over the Pier Line, Over a Beam, Bottom Gage.

Fig 4.7 -Raw and corrected Strain-Clark County (CLA-70-0659)– Phase I, at Midspan Between Girder Lines, Top Gage.
Fig 4.8 Raw Strain and Corrected Strain – Darke County (DAR-127-1977) Over Pier Line, Over Beam, Top Gage.

Fig 4.9-Raw-Corrected Strain – Darke County (DAR-127-1977) at Midspan, Between Girder Lines, Bottom Gage.
The following observations can be made from the raw-corrected strain graphs.

- There is a sudden increase in strains in the gages placed between the girder lines. This sudden increase in the strains occurs at approximately 100 days from the day the deck is cast.

- There is a sudden drop in the gages placed over the beams. This sudden decrease in strains occurs approximately at around 100 days from the day the deck is cast.

- There is no clear connection between these changes and any observed event.

### 4.6 STRESS PRODUCING STRAIN

**Figure 4.10 – Effect of Restraint on a Block.**

Consider the concrete block shown in Figure 4.10, which is attempting to deform. Assume a strain gage is attached to the block. If concrete block is allowed to deform freely, a strain will be recorded in the gage, but there is no restraint so the stress will be zero. If the concrete block is completely restrained, the strain recorded in the strain gages
will be zero but a stress will develop. If the concrete block is partially restrained, then the strains recorded in the strain gages are inversely proportional to extent of restraint on the concrete block while the stresses are proportional. The lower the restraint is on the block, the higher the strain and the lower the stress.

In bridge deck concrete, the deformations come from forces, shrinkage and temperature. The force related strains come from dead loads, live loads and restraint loads. Since this is an unshored system, the deck does not carry any self weight dead load. The only dead loads on the deck would be due to the parapets and this will not cause significant dead load strains. The live load on the deck is dynamic in nature and the VWG will not measure any strains from these loads as they do not respond fast enough (see Chapter 3). Thus, the only stress related strain the gage will measure is due to restraint of the deck.

The gage will also measure contractions due to creep and shrinkage. Creep is caused by constant stresses and the only constant stress is due to the parapets. This stress is negligible so the creep effects are also likely to be negligible. Thus, only shrinkage strains are significant.

It can be shown (Mitchell and Collins, 1991, p. 191) that the strain due to stress is given by:

\[ \varepsilon_{\text{stress}} = \varepsilon_{\text{total}} - \varepsilon_{\text{shrinkage}} - \varepsilon_{\text{temperature}} \]

where:
\[ \varepsilon_{\text{stress}} = \text{strain due to stress} \]
\[ \varepsilon_{\text{total}} = \text{total strain} \]
\[ \varepsilon_{\text{shrinkage}} = \text{free shrinkage strain} \]
\[ \varepsilon_{\text{temperature}} = \text{temperature strain} \]
The sign convention is that expansive/tensile strain is POSITIVE. The shrinkage strain in the equation is the free shrinkage strain which would be measured if the concrete were completely unrestrained. Since shrinkage strains are contractions, the shrinkage strain is always negative. Temperature strains can be expansions when the temperature rises, but since the datum is the peak temperature of hydration, the strains are usually contractions as well. As previously shown, the measured, corrected VWG readings include the temperature strain. Thus, the equation becomes:

\[ \varepsilon_{\text{stress}} = \varepsilon_{\text{measured}} - \varepsilon_{\text{shrinkage}} \]

where:

- \( \varepsilon_{\text{stress}} \) = strain due to stress
- \( \varepsilon_{\text{measured}} \) = measured strain
- \( \varepsilon_{\text{shrinkage}} \) = free shrinkage strain

Note that the equation works regardless of the level of restraint against shrinkage. If the concrete is unrestrained, the measured strain will equal the free shrinkage strain, so the strain due to stress (restraint) will be zero. If the concrete is completely restrained, the measured strain will be zero, and the strain due to restraining stresses will equal the free shrinkage strain (as would found from a strength of materials solution). If the restraint were X%, the measured strain would be (1-X%) times the free shrinkage strain and the strain due to restraining stress would be X% times the free shrinkage strain.

4.7 SHRINKAGE IN CONCRETE:

There are three basic types of shrinkage: Plastic, autogeneous and drying shrinkage.

Plastic shrinkage occurs during the first few hours after pouring fresh concrete. Exposed surfaces of the bridge deck are affected by the exposure of dry air because of
their large contact surface. Moisture evaporates faster from the concrete surface than it is replaced by the bleed water from the lower layers of the concrete slab.

Autogeneous shrinkage occurs during hydration. When concrete has a very high cement content, the heat of hydration will be high. If that same concrete has a low water/cementitious material ratio, it is possible to literally boil the water out of the concrete. This would cause shrinkage during the first 24 hours. This effect is usually missed because the standard AASHTO test for shrinkage (T160) doesn’t begin until the specimen is 24 hours old and can be demolded.

Drying shrinkage occurs after the concrete has already attained its final set and the hydration of concrete is almost complete. Drying shrinkage is the decrease in volume of concrete. The phenomenon which results in the increase of the volume of concrete due to water absorption is called swelling. Swelling and shrinkage are essentially movement of water into or out of the gel structure of concrete due to the difference in humidity or saturation levels between the concrete and the surroundings.

Several factors affect the magnitude of drying shrinkage:

1. Aggregate: The aggregate present in concrete restrains the shrinkage of the cement paste since the aggregates do not shrink. Aggregates with rough surfaces provide more resistance to shrinkage. Hence the degree of restraint of a given concrete depends on the type of aggregate.

2. Water Content: The higher the water content, the larger the shrinkage.

3. Cement Content: Since only the cement shrinks, higher cement contents cause more shrinkage.

4. Size of the concrete element: Both the magnitude and rate of shrinkage increase with an increase in the volume of the concrete element. However, the duration of
shrinkage is longer if the element size is larger as more time is required for drying shrinkage of interior regions.

5. Ambient conditions: The relative humidity of the surroundings affects the magnitude of the shrinkage. The rate of shrinkage is lower at regions of higher relative humidity.

6. Admixtures: This effect varies depending on the type of admixture. An accelerator such as calcium chloride accelerates the setting of the concrete, increases the shrinkage.

7. Amount of reinforcement. The reinforcement present in the concrete restrains shrinkage. But restraining shrinkage results in development of stresses and may lead to curvature and deflections in case of unsymmetrical reinforcement.

8. Type of cement: Different cements have different shrinkage characteristics.

9. Carbonation: This is the type of shrinkage that occurs due to the reaction between carbon dioxide present in the atmosphere and that present on the paste. Shrinkage can cause tensile strain due to differential effects. Even in an unrestrained system, the outer skin of the concrete will shrink more than the core. This causes a tension on the skin and a compression in the core.

As part of this experimental program, AASHTO T160 shrinkage tests were conducted. For bridge DAR-127, the standard test was performed using a comparometer. There was some problem with this test as the results were found to be nonsense. For the CLA-70 bridges, the test was conducted by embedding vibrating wire strain gages into the blocks. This allowed measurement of shrinkage from the time the concrete was plastic. As required by T160, the blocks were cured after de-molding. Since the decks were water cured for 7 days, the specimens were water cured for 7 days. All tests were
performed at 50% RH and 73 degrees F as required. Figure 4.11 shows the shrinkage of a typical specimen from the Clark County Bridge (the gap in the data is due to a data collection system failure).

![Graph showing shrinkage strain over time for a typical specimen from the Clark County Bridge.](image)

**Fig 4.11 Shrinkage Strains – Clark County County (strain given in microstrain).**

Since the tests for the Darke County bridge were not usable, the shrinkage strain was assumed to be 1000 microstrain and the strain curve was assumed to follow the formula:

\[ \varepsilon_{sh} = \frac{t}{35 + t} \times 1000 \]

as given in ACI Report 209. The term “t” is time in days.

**4.8 EVALUATION OF STRESS PRODUCING STRAIN**

Shown are some typical results from the study. A complete set of graphs which include all data are in Appendix B. The graphs shown below include the following:
• Free Thermal Strain = This is the theoretical thermal strain which would be found in the concrete in absence of any restraint.

• Total Strain – This is the strain which would be measured by some external device – $\varepsilon_{\text{ext}}$.

• Total Free Strain- The theoretical strain in the concrete in absence of restraint due to the effects of shrinkage and temperature.

• Stress Producing Strain – The strain in the concrete associated with stress. This may be due to load, restrained shrinkage or restrained thermal movement.

• Corrected raw strain – the strain reading as taken in the field, corrected only for the difference in co-efficient of thermal expansion between the concrete and the steel gage.

Fig 4.12 Clark County – CLA-70-0659 – Phase 1, at Pier Line, Over a Beam, Bottom Gage.
Fig 4.13 - Clark County, CLA – 70-0659, Phase 1, at Midspan, Between Girder Lines, Top Gage.

Fig 4.14 Darke County, DAR-127-1977- Phase 1, at Pier Line, Over Beam, Top Gage.
Fig 4.15 – Darke County, DAR-127-1977 - Phase I, at Midspan, Between Girder Lines, Bottom Gage.

The following observations can be made in general about trends of various strains presented above:

- The actual strains follow the trends of the free thermal strains in the gages over the beams.
- The actual strains follow the trend of the free thermal strain for an initial period in the gages in the middle. The actual strains in concrete seem to suddenly increase and go into tension after a period of around 50 to 100 days.
- The stress producing strain follows the pattern of the shrinkage curve. The trends in stress producing strain closely match those of the corrected strain. But after the shrinkage curve flattens out, the stress producing strain reaches a constant value.
It is important to note that actual shrinkage strain of the deck is NOT known. The free shrinkage strain used in the Clark County graphs is that measured by AASHTO T160, where the specimen is dried under the extreme condition of 50% relative humidity. For Darke County, it is an assumed shrinkage function, as previously described. Thus, the stress producing strain (restraint strain) shown is an upper bound. The actual stress producing strain is probably less. In retrospect, it would have been useful to put shrinkage specimens on the bridge site to measure field free shrinkage.

4.9 ESTIMATING THE DEGREE OF THERMAL RESTRAINT

The degree of restraint against thermal strains can be estimated by the following procedure: The effect of shrinkage will be minimal after 150 days. The strain created by the dead load is constant and does not vary with time or temperature. During this phase, any change in the strain recorded by the vibrating strain gage can be attributed to corresponding change in temperature.

When a concrete block is allowed to expand or contract freely, unit expansion or contraction per unit rise or fall in temperature will be equal to its thermal coefficient. Bridge decks are usually allowed movements to relieve the temperature and shrinkage stresses. The extent of restraint the steel girder offers to the concrete slab can be determined from how much the concrete is allowed to expand for a unit rise in temperature, after 150 days from the day of the pour.

Degree of restraint = 1 - \( \frac{\alpha_{\text{con-allow}}}{\alpha_{\text{concrete}}} \)

\( \alpha_{\text{con-allow}} \) is the average unit change in strain per unit rise in temperature

\( \alpha_{\text{concrete}} \) is the thermal coefficient for concrete (5.77 x 10^{-6} / °F).

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The degree of restraint for various positions of the gages for the four phases of the Clark County bridges are summarized in Table 4.1-4.4 and are summarized in Tables 4.5 and 4.6 for the two phases of the Darke County Bridge.

<table>
<thead>
<tr>
<th>Location of the Gage</th>
<th>Degree of Restraint (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Over the beam at the pier line, bottom of slab</td>
<td>16.70</td>
</tr>
<tr>
<td>Over the beam at the pier line, top of slab</td>
<td>10.22</td>
</tr>
<tr>
<td>Between girder lines, at the pier line, bottom of slab</td>
<td>5.54</td>
</tr>
<tr>
<td>Between girder lines, at the pier line, top of slab</td>
<td>5.74</td>
</tr>
<tr>
<td>Midspan, over beam, bottom of slab</td>
<td>12.38</td>
</tr>
<tr>
<td>Midspan, over beam, top of slab</td>
<td>7.15</td>
</tr>
<tr>
<td>Midspan, Between girder lines, bottom of slab</td>
<td>14.25</td>
</tr>
<tr>
<td>Midspan, Between girder lines, top of slab</td>
<td>9.73</td>
</tr>
</tbody>
</table>

**Table 4.1 – Clark County Bridge CLA-70-0659 - Phase I**

<table>
<thead>
<tr>
<th>Location of the Gage</th>
<th>Degree of Restraint (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Over the beam at the pier line, bottom of slab</td>
<td>8.13</td>
</tr>
<tr>
<td>Over the beam at the pier line, top of slab</td>
<td>7.95</td>
</tr>
<tr>
<td>Between girder lines, at the pier line, bottom of slab</td>
<td>7.22</td>
</tr>
<tr>
<td>Between girder lines, at the pier line, top of slab</td>
<td>10.15</td>
</tr>
<tr>
<td>Midspan, over beam, bottom of slab</td>
<td>10.58</td>
</tr>
<tr>
<td>Midspan, over beam, top of slab</td>
<td>10.95</td>
</tr>
<tr>
<td>Midspan, Between girder lines, bottom of slab</td>
<td>9.82</td>
</tr>
<tr>
<td>Midspan, Between girder lines, top of slab</td>
<td>11.94</td>
</tr>
</tbody>
</table>

**Table 4.2 – Clark County Bridge CLA-70-0659 – Phase 2**
<table>
<thead>
<tr>
<th>Location of the Gage</th>
<th>Degree of Restraint (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Over the beam at the pier line, bottom of slab</td>
<td>20.01</td>
</tr>
<tr>
<td>Over the beam at the pier line, top of slab</td>
<td>10.19</td>
</tr>
<tr>
<td>Between girder lines, at the pier line, bottom of slab</td>
<td>18.72</td>
</tr>
<tr>
<td>Between girder lines, at the pier line, top of slab</td>
<td>9.43</td>
</tr>
<tr>
<td>Midspan, over beam, bottom of slab</td>
<td>17.82</td>
</tr>
<tr>
<td>Midspan, over beam, top of slab</td>
<td>10.67</td>
</tr>
<tr>
<td>Midspan, Between girder lines, bottom of slab</td>
<td>12.62</td>
</tr>
<tr>
<td>Midspan, Between girder lines, top of slab</td>
<td>NA</td>
</tr>
</tbody>
</table>

Table 4.3 – Clark County Bridge CLA-70-0659 - Phase 3

<table>
<thead>
<tr>
<th>Location of the Gage</th>
<th>Degree of Restraint (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Over the beam at the pier line, bottom of slab</td>
<td>NA</td>
</tr>
<tr>
<td>Over the beam at the pier line, top of slab</td>
<td>13.51</td>
</tr>
<tr>
<td>Between girder lines, at the pier line, bottom of slab</td>
<td>8.90</td>
</tr>
<tr>
<td>Between girder lines, at the pier line, top of slab</td>
<td>18.43</td>
</tr>
<tr>
<td>Midspan, over beam, bottom of slab</td>
<td>16.83</td>
</tr>
<tr>
<td>Midspan, over beam, top of slab</td>
<td>14.70</td>
</tr>
<tr>
<td>Midspan, Between girder lines, bottom of slab</td>
<td>12.98</td>
</tr>
<tr>
<td>Midspan, Between girder lines, top of slab</td>
<td>14.78</td>
</tr>
</tbody>
</table>

Table 4.4 – Clark County Bridge CLA-70-0659 – Phase 4
The degree of thermal restraint varies from approximately 10-20%. There does not appear to be any specific pattern. In general, the Clark County bridges show less restraint. This is probably attributable to the bearings. The Clark County bridges had
elastomeric pads under the bearings and slotted connections in the bearing plates. This allowed the bridge to move more freely.

It is important to note that the results in Tables 4.1-4.6 include other movements with the thermal restraint. For example, when the bridge heats up, the top surface is hotter than the bottom, causing a camber of the beams. This would be read in the strain used to calculate restraint and is not easily separable. The point of Tables 4.1-4.6 is to provide a magnitude of thermal restraint – e.g. the restraint is approximately 20% not 50% or 90%.

These numbers show the extent to which temperature may contribute to cracking. Recall that the slab sets near the time of peak heat of hydration, usually around 130-150°F. Since future slab temperatures will be below this, the slab will always be in a state of contraction due to temperature. If free to move, the slab would simply contract, but because the slab is partially restrained, part of the contraction strain will translate to tensile strain. Consider a slab which sets with a temperature of 140°F. As some later time, it cools to 0°F. If unrestrained, the slab would contract by:

\[ 140°F \times 5.5 \text{ microstrain/°F} \times 0.8 = 770 \text{ microstrain.} \]

Since the slab is actually 20% restrained, the free strain is 770 x 0.8 = 616 microstrain and the remaining 154 microstrain is tensile strain which develops due to restraint. 150 microstrain is approximately the tensile cracking strain of the concrete, so temperature restraint can easily crack a deck.

It is also important to note that these restraint values are not valid for shrinkage. When the bridge experiences thermal movements, all of the bridge elements move in some way. Shrinkage only affects the deck; the beams do not shrink. As a result, the degree of shrinkage restraint will be greater.
4.10 DISCUSSION OF THE RESULTS

The data shows that a tensile strain forms immediately in the deck due to contraction of the deck. The deck takes its final set while still hot from the heat produced when the concrete hydrates. As the deck cools, it contracts. Since the deck is restrained by the girders, tensile strains develop. The tensile strains are on the order of 100-200 microstrain. For a normal deck, the early age modulus of elasticity would be approximately 1,000,000 to 2,000,000 psi. The tensile stress would be 100-200 psi. This is significant since the concrete is likely to have a low tensile strength at early ages. Even at later ages, the tensile strength is likely to be 500-800 psi, so 100-200 psi represents a significant portion of the available strength.

Shrinkage is also significant. It was expected that the VWG would show a decrease in strain (contraction), however, this decrease would not be as large as shrinkage strain. The difference is the stress producing strain. This behavior was observed with some gages, especially those over the beams (Figure 4.12). However, in some gages the measured strains do not appear vary much at all and seem to simply follow the thermal movements. This would seem to indicate complete restraint against shrinkage. In other gages tensile strains are measured and this was totally unexpected. It was expected that the only tensile stresses which would develop would be due to restraint of deck. In this case, the VWG would still show a contraction but just not as much as the free shrinkage contraction. The cause of these tensile strains is not clear. One possible explanation is that they tend to occur in gages on the bottom of the slab between the girder lines. Here the gages are below the bottom rebar mat and are in the cover concrete. The concrete between the girder lines is completely exposed and would have the highest shrinkage. It
is possible that this cover concrete is trying to shrink and has developed some microcracking. The gage may be measuring the opening of these microcracks.

It is also interesting to note that in each phase, there is a place where the strains begin to diverge. For the Clark County bridges, the point is near 100 days. For Darke County, it is near 50 days. There is no specific event which can be connected with this date which would cause this divergence.

Finally, there is the question of why the Clark County bridges cracked and the Darke County bridge did not, especially when Figures 4.12-4.15 seem to show that the Darke County bridge develops higher strains than the Clark County bridges. There are some possible reasons:

1) Figures 4.12-4.15 use the measured free shrinkage from AASHTO T160. This is an extreme drying condition (50% RH). As a result, Figures 4.12-4.15 represent worst cases for tensile stress development. The actual bridge decks probably did not have this level of drying. In fact, the Darke County bridge has a more favorable condition. This bridge is a low bridge over water. It is probable that the waterway increases the RH and limits the shrinkage. The Clark County bridge is a high bridge, on an interstate over a railway. The Clark County bridge is subject to more wind (therefore more drying) and has no source of moisture.

2) There was a delay of 8 months between phases of the Clark Bridges. When the 2 phases of each bridge were connected together, the earlier phase would have provided a high level of restraint since earlier phase had already undergone most of its shrinkage. In Darke County, the two phases
were completed within 2 month of each other so the restrain of the first phase would be less.

3) The Clark County bridge had a bearing problem. The bridge was jacked up at the ends and the bearings were replaced. The cracking was noted after this operation. It is uncertain if this had anything to do with the deck cracking.
CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

5.1 SUMMARY

This project attempted to determine the cause of cracking in bridge decks. Data from bridge decks was analyzed to determine if the cause could be found. Three bridge decks were instrumented to measure the strains in the decks. It was found that deck cracking is due to a combination of various types of shrinkage and thermal effects. While the emphasis of this report is on transverse cracking, the conclusions apply equally to longitudinal cracking in decks.

5.2 CONCLUSIONS

1) ODOT supplied data on approximately 100 bridge decks in the state. This data included current deck condition and various parameters measured when the decks were cast. Weak correlations were found between deck cracking and slump, shrinkage, chloride permeability, deck strength and time of year when the deck was cast. A weak correlation with geographic area was attributed to subjective difference in deck rating when inspection of sample bridges showed little difference in deck condition. No single factor could be found which explained deck cracking.

2) District 12 claimed that using high absorption coarse aggregates (absorptions > 1%) prevented deck cracking. Using data supplied by ODOT for decks statewide, it was found that decks which did not crack had aggregate absorptions > 1%, but many other decks with aggregate absorptions > 1% still cracked. Therefore, using a high absorption aggregate appears to help, but it does not
completely solve the problem. It is thought that using a high absorption aggregate helps to mitigate autogeneous shrinkage by providing a source of moisture while the concrete is setting.

3) Data from a National Co-operative Highway Research Program study on continuous bridges (NCHRP Report 519; 2004) showed that deck behavior should be considered from the time the deck is at or near the peak temperature caused by heat of hydration. The deck sets near the peak temperature caused by the heat hydration. After this, the deck attempts to contract due to cooling but is restrained by the girder beneath. This can cause tensile stress to occur. In the deck instrumented in the study, it was found that this contraction can cause tensile strains of 100 to 200 microstrain. Assuming the normal modulus of elasticity of deck concrete can range from 3,000,000 to 5,000,000 psi, the tensile stresses can range from 300 to 1000 psi. Additional thermal, tensile stresses can occur due to normal contractions of the deck when the ambient temperature drops.

4) Drying shrinkage of the deck increases tensile stress. As the deck attempts to shrink, the girders restrain it, generating tensile stresses. It is difficult to quantify this as the exact shrinkage in the deck is unknown. However, since the deck is already in tension due to thermal effects, any shrinkage which occurs simply adds to the tensile strain. It is important to remember that shrinkage will not be uniform. Exposed surfaces will shrink more than unexposed areas, such as concrete in contact with the girder or concrete in the center of the slab. This differential shrinkage will set up tensile stresses as one part of the slab attempts to shrink more than others.
5) Autogeneous shrinkage occurs when a high heat of hydration literally boils the water out of the concrete. This occurs in concrete with high cement contents. This may contribute to early age cracking of decks.

6) Plastic shrinkage, caused by loss of water due to evaporation during the casting phase, may also contribute to deck cracking. Plastic shrinkage can be significant in pozzolanic concrete which has very little bleed water to protect the concrete surface.

Conclusions 3-6 are in substantial agreement with studies on deck cracking performed in other states, including Georgia, Utah, Colorado, New Jersey and Indiana.

5.3 SPECIFIC RECOMMENDATIONS

1) Reduce cement contents to lowest level practical, but which will still provide sufficient strength. Current ODOT deck concretes (class S or HPC) use cementitious material content of 700#/ cubic yard. Lowering the cement content has three beneficial results. Lower cement contents will have lower peak temperatures due to heat of hydration. This will reduce the tensile strains caused by the deck cooling after final set and will mitigate autogeneous shrinkage cause by high heat of hydration literally boiling the water out of the concrete. Concrete with a low cement content will have a low shrinkage potential since only the cement phase shrinks. Finally, costs will be reduced since cement is the most expensive part of the concrete. This item can be easily addressed with performance based specifications.

2) Add retarders. Retarders, either chemical or mineral (fly ash), reduce peak temperatures due to heat of hydration. Using fly ash will also help by
reducing permeability. Some ODOT mixes already use retarders and/or fly ash.

3) Use larger aggregates. Large aggregates decrease cement demand and mitigate drying shrinkage. However, using large aggregates will limit maximum strength potential. This should not be a problem as high strength is not needed in decks.

4) Pay close attention to placement and curing. Plastic shrinkage (where the surface of fresh concrete dries due to evaporation) can lead to cracking in decks. Plastic shrinkage is especially problematic on high performance mixes which use fly ash or silica fume as these mixes do not have a large amount of bleed water to protect the surface from drying. Strict enforcement of Item 511.10 in the ODOT Construction and Materials Specifications is necessary. Concrete should also be properly cured. Water curing is preferred as this is shown to limit drying shrinkage potential. For concrete containing silica fume or fly ash, water curing is essential. The longer the water cure, the more the shrinkage potential is reduced, but long curing periods may not be practical, especially with accelerated construction schedules.

5) Keep water/cementitious material (w/c) ratios as high as possible while retaining the required strength and durability. Concrete with a high cement content and low w/c ratios are subject to autogeneous shrinkage. Autogeneous shrinkage is where a high heat of hydration (due to a high cement content) will cause excess evaporation of the water in the concrete. The results in an early age shrinkage of the concrete. Use of a high absorptive
aggregate (absorption > 1%) may also help by providing a source of water for proper strength development.

6) Consider the use of shrinkage reducing admixtures to limit drying shrinkage.

7) Consider reducing the amount of reinforcing steel in the bridge deck. Concrete shrinks but the reinforcing steel does not. This creates tensile stresses in the concrete from the restraint. Elementary mechanics shows that the degree of restraint is proportional to the volume fraction of the steel. The 3rd Edition of the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design Specifications (LRFD) for Bridges (2003) has an empirical design method for decks which uses far less steel than ODOT is currently using. Reducing the amount of steel in the deck would help to reduce cracking and result in a cost savings.

The recommendations provided above are in the lines of those suggested by previous research done in the area of transverse cracking. Similar recommendations to the problem of transverse cracking were provided by NCHRP Synthesis Report 333(2004). Griffin, Khan and Lopez (2004) had provided similar recommendations for mitigating transverse deck cracking in the composite deck over Interstate 75 in Henry County, Georgia. Linford and Reaveley (2004) provided recommendations on the same lines to minimize transverse cracking of bridge decks as part of reconstruction of I-15 in Utah. Studies from Colorado and New Jersey made similar recommendations.
5.4 ITEMS NOT COVERED IN THIS PROJECT AND RECOMMENDATIONS FOR FUTURE RESEARCH

The three bridges instrumented in this study did not use integral or semi-integral abutments. Such abutments increase the restraint on the bridge deck and may increase cracking potential. However, since no integral abutment bridges were instrumented, the absolute degree of restraint and the relative degree of restraint as compared to the bridges in this study is not known. A project to evaluate the level of restraint in integral and semi-integral bridges may be warranted.
CHAPTER 6

REFERENCES


APPENDIX A

Bridge Plans and Instrumentation

A 1 Pictures of the bridge, bridge plan and placement of the gages.

Fig A1 View of the girders, cross bracing and columns
Fig A2 Clark County Bridge- Deck View

Fig A-3 Transverse crack in the deck-Clark County

Fig A4 Criss-Cross cracks at midspan- Clark County
Fig A5 Framing Plan Instrument Placement-Clark County

Fig A6-Framing Plan-Darke County

FRAMING PLAN

* POSITION OF VIBRATING WIRE GAGES
Appendix B Strains

B 1 Raw strains – All Gages-Phase 1-4. – Clark County

Pier Beam Bottom

Pier Top beam
B-2 Total Free Strain – All Gages Phase 1-4 – Clark county
B-5 Correction of raw strain data for the difference in thermal coefficients of the steel of the vibrating strain gage and the concrete.—Clarke County-Phase 1
B-6 Representation of the various strain – Clark County-Phase 1

Over Pier, Over Beam, Bottom Gage Phase I CLA 70

Over Pier, Over Beam, Top Gage, Phase I, CLA-70
Midspan Over Beam, Bottom Gage, Phase I, CLA-70

Strain
Stress Producing Strain
Corrected Raw Strain
Free Thermal Strain
Strain Measured by an External Device
Total Free Strain

Days From Casting Phase I

Midspan, Over Beam, Top Gage, Phase I, CLA-70

Strain
Stress Producing Strain
Corrected Raw Strain
Strain Measured by an External Device
Free Thermal Strain
Total Free Strain

Time From Casting Phase I
B-7 Correction of raw strain data for the difference in thermal coefficients of the steel of the vibrating strain gage and the concrete.-Darke County-Phase 1
B-8 Representation of the various strain – Darke County-Phase 1

Over Pier, Over Beam, Bottom Gage, Phase I, DAR-127

Over Pier, Over Beam, Top Gage, Phase I, DAR-127