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Bo Kong
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THERMAL FIELD INVESTIGATIONS AND APPLICATIONS TO INTEGRAL ABUTMENT BRIDGES WITH FRP PANELS

A Dissertation

Submitted to the Graduate Faculty of the Louisiana State University and Agricultural and Mechanical College in partial fulfillment of the requirements for the degree of Doctor of Philosophy

in

The Department of Civil and Environmental Engineering

by

Bo Kong
B.S., Dalian University of Technology, P. R. China, 2008
December 2012
DEDICATION

To my parents and girlfriend
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# TABLE OF CONTENTS

ACKNOWLEDGEMENTS .......................................................................................................................... iii

LIST OF TABLES ........................................................................................................................................ vii

LIST OF FIGURES ........................................................................................................................................ ix

ABSTRACT .................................................................................................................................................... xiv

CHAPTER 1.  INTRODUCTION .......................................................................................................................... 1
  1.1. Thermal Effects of Bridges .................................................................................................................. 1
  1.2. Study of Integral Abutment Bridges .................................................................................................. 3
  1.3. Study of FRP Panel Bridges .............................................................................................................. 5
  1.4. Overview of the Dissertation ............................................................................................................ 5
  1.5. References .......................................................................................................................................... 8

CHAPTER 2.  TEMPERATURE DISTRIBUTION BEHAVIORS OF GFRP HONEYCOMB
HOLLOW SECTION SANDWICH PANELS ..................................................................................................... 12
  2.1 Introduction ........................................................................................................................................... 12
  2.2 Heat Transfer Mechanism .................................................................................................................... 14
    2.2.1 Heat Transfer Equation .................................................................................................................. 14
    2.2.2 Boundary Conditions ...................................................................................................................... 15
    2.2.3 Environmental and Thermal Parameters ....................................................................................... 15
  2.3 Field Monitoring Study ....................................................................................................................... 16
    2.3.1 Project Description ......................................................................................................................... 16
    2.3.2 Monitoring Results ......................................................................................................................... 17
  2.4 Numerical Modeling Study ................................................................................................................... 22
    2.4.1 GFRP Panel Thermal Properties Prediction ................................................................................... 23
    2.4.2 Modeling Method ............................................................................................................................ 24
    2.4.3 Comparison of Results .................................................................................................................. 25
  2.5 Parametric Study ................................................................................................................................... 27
    2.5.1 Air Convection ............................................................................................................................... 28
    2.5.2 Mutual Radiation .............................................................................................................................. 30
    2.5.3 Environmental and Material Properties ......................................................................................... 33
  2.6 Conclusion .......................................................................................................................................... 34
  2.7 References .......................................................................................................................................... 35

CHAPTER 3.  TEMPERATURE AND STRESS DISTRIBUTIONS OF GFRP SANDWICH PANEL
BRIDGES VERSUS CONCRETE AND STEEL BRIDGES ........................................................................ 38
  3.1 Introduction .......................................................................................................................................... 38
  3.2 Temperature Distribution Modeling Method ......................................................................................... 40
  3.3 Parametric Study ................................................................................................................................... 43
    3.3.1 Predication of GFRP Panel Properties ........................................................................................... 45
    3.3.2 Discussion of Temperature Gradients ............................................................................................ 50
  3.4 Discussion of Thermal Strain and Stress Results ................................................................................... 56
    3.4.1 Thermal Strain Results .................................................................................................................. 58
    3.4.2 Thermal Stress Results .................................................................................................................. 60
  3.5 Conclusion .......................................................................................................................................... 62
  3.6 References .......................................................................................................................................... 64
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.3.1</td>
<td>Model of IAB</td>
<td>151</td>
</tr>
<tr>
<td>7.3.2</td>
<td>Prediction of GFRP Panel Properties</td>
<td>152</td>
</tr>
<tr>
<td>7.3.3</td>
<td>Verification of GFRP Panel Properties</td>
<td>155</td>
</tr>
<tr>
<td>7.4</td>
<td>Numerical Study Results</td>
<td>156</td>
</tr>
<tr>
<td>7.4.1</td>
<td>IAB Project Study- Base Case</td>
<td>156</td>
</tr>
<tr>
<td>7.4.2</td>
<td>Verification of Stiffness-Equivalent Method</td>
<td>160</td>
</tr>
<tr>
<td>7.4.3</td>
<td>GFRP Panel Applications</td>
<td>162</td>
</tr>
<tr>
<td>7.5</td>
<td>Conclusion</td>
<td>165</td>
</tr>
<tr>
<td>7.6</td>
<td>References</td>
<td>167</td>
</tr>
<tr>
<td>8.1</td>
<td>Thermal Behavior of FRP Panel Bridges</td>
<td>169</td>
</tr>
<tr>
<td>8.2</td>
<td>Field Monitoring Study of Integral Abutment Bridges</td>
<td>171</td>
</tr>
<tr>
<td>8.3</td>
<td>Numerical Modeling Study of Integral Abutment Bridges</td>
<td>172</td>
</tr>
<tr>
<td>8.4</td>
<td>Applications of FRP Panels on Integral Abutment Bridges</td>
<td>173</td>
</tr>
<tr>
<td>8.5</td>
<td>Recommendations for Future Research</td>
<td>174</td>
</tr>
<tr>
<td>8.6</td>
<td>References</td>
<td>175</td>
</tr>
<tr>
<td>VITA</td>
<td></td>
<td>176</td>
</tr>
</tbody>
</table>
LIST OF TABLES

Table 2-1 Constituents and Laminates Material Properties* .......................................................... 24
Table 2-2 FEM Parameters .................................................................................................................. 25
Table 2-3 Temperature Distribution at 12:00, June 23 ..................................................................... 34
Table 3-1 FEM Parameters from Elbadry and Ghali (1983)................................................................. 41
Table 3-2 Parameters Used in FEM .................................................................................................... 44
Table 3-3 AASHTO Concrete Beam Section Properties (cm) ............................................................ 45
Table 3-4 Steel Girder Section Properties (cm) .................................................................................. 45
Table 3-5 GFRP Panel Constituent and Laminate Thermal Properties* ........................................... 46
Table 3-6 GFRP Panel Elastic Properties (Oghumu 2005) ................................................................. 46
Table 3-7 FEM Parameters for Crawford County Bridge ................................................................. 47
Table 3-8 Deflection Comparisons of the FEM and Experiment Results ......................................... 49
Table 3-9 Strain Comparisons of the FEM and Experiment Results ............................................... 50
Table 3-10 Temperature Comparisons for Concrete Beams with GFRP and Concrete Slabs ........ 56
Table 3-11 Temperature Comparisons for Steel Girders with GFRP and Concrete Slabs ............... 56
Table 4-1 Properties of GFRP Panel Bridge (Oghumu 2005) ........................................................... 74
Table 4-2 Measured Temperature Loads for Numerical Modeling .................................................... 78
Table 4-3 Absolute Results for the GFRP Panel Bridge .................................................................... 78
Table 4-4 Normalized Results for the GFRP Panel Bridge ............................................................... 78
Table 4-5 AASHTO Concrete Girders Section Properties (cm) ......................................................... 80
Table 4-6 Steel Beam Section Properties (cm) ................................................................................... 81
Table 4-7 Concrete Slab and Concrete Beam Cases Absolute Results ........................................... 82
Table 4-8 Concrete Slab and Concrete Beam Cases Normalized Results ........................................ 83
Table 4-9 GFRP Slab and Concrete Beam Cases Absolute Results ............................................... 83
Table 4-10 GFRP Slab and Concrete Beam Cases Normalized Results ............................................ 83
Table 4-11 Concrete Slab and Steel Girder Cases Absolute Results .......................................................... 83
Table 4-12 Concrete Slab and Steel Girder Cases Normalized Results ......................................................... 83
Table 4-13 GFRP Slab and Steel Girder Cases Absolute Results ................................................................. 84
Table 4-14 GFRP Slab and Steel Girder Cases Normalized Results ............................................................... 84
Table 5-1 Lists of the Field Monitoring Programs Reported in the Literature .............................................. 94
Table 5-2 Instrumentations Applied on Caminada Bay Bridge ...................................................................... 97
Table 5-3 Normality Test of the Strain and Temperature Readings of Strain Gage at Slab5 ....................... 102
Table 6-1 Lists of Numerical Studies in Literature ......................................................................................... 124
Table 6-2 Parametric Study Cases .............................................................................................................. 139
Table 7-1 Properties of GFPR Slab and Equivalent Slab ............................................................................. 154
Table 7-2 Comparisons between GFRP and EQUIV Slab with the X- Dir. Supports Fixed ..................... 155
Table 7-3 Comparisons between GFRP and EQUIV Slab with the Y- Dir. Supports Fixed ..................... 155
Table 7-4 Forces at Slab Section under Free Support ..................................................................................... 161
Table 7-5 Forces at Slab Section under Fixed Support .................................................................................. 162
Table 7-6 Strength of GFRP materials (http://en.wikipedia.org/wiki/Fiberglass) ........................................ 165
LIST OF FIGURES

Fig. 2-1 GFRP Honeycomb Hollow Section Sandwich Panel Bridge .................................................. 17
Fig. 2-2 Measured Temperature from January 24 to July 13, 2004 .................................................... 18
Fig. 2-3 Measured Temperature from June 21 to June 27, 2004 ...................................................... 18
Fig. 2-4 Measured Temperature from Feb. 7 to Feb. 14, 2004 ............................................................. 19
Fig. 2-5 Environmental Conditions from Field Measurements and Weather Station .......................... 19
Fig. 2-6 Temperature Linear Fitting Results of the Bottom Surface and the Air ................................. 20
Fig. 2-7 Temperature Linear Fitting Results of the Top Surface and the Air ....................................... 20
Fig. 2-8 GFRP Hollow Section Sandwich Panel and Core Configuration ........................................... 23
Fig. 2-9 Sketch of the GFRP Panel Finite Element Model ................................................................. 25
Fig. 2-10 Comparison of Top Surface Temperature between Modeled and Measured Results .......... 26
Fig. 2-11 Comparison of Bottom Surface Temperature between Modeled and Measured Results ....... 26
Fig. 2-12 Modeled Thermal gradients on June 23, 2004 .................................................................... 27
Fig. 2-13 Absolute Thermal Gradients under Convection Effects at 12:00, June 23 ............................ 29
Fig. 2-14 Normalized Thermal Gradients under Convection Effects at 12:00, June 23 ....................... 29
Fig. 2-15 Absolute Thermal Gradients under Convection Effects at 22:00, June 23 ......................... 30
Fig. 2-16 Normalized Thermal Gradients under Convection Effects at 22:00, June 23 ....................... 30
Fig. 2-17 Absolute Thermal Gradients under Radiation Effects at 12:00, June 23 .............................. 31
Fig. 2-18 Normalized Thermal Gradients under Radiation Effects at 12:00, June 23 ....................... 32
Fig. 2-19 Absolute Thermal Gradients under Radiation Effects on 22:00, June 23 ........................... 32
Fig. 2-20 Normalized Thermal Gradients under Radiation Effects at 22:00, June 23 ....................... 33
Fig. 3-1 Thermal Gradients from ANSYS Prediction and Reference by Elbadry and Ghali (1983) ........ 42
Fig. 3-2 Bridge Temperatures for All Iterations from ANSYS Prediction ........................................... 43
Fig. 3-3 Concrete and Steel Girders Geometrical Configurations .................................................... 45
Fig. 3-4 Sketch of the GFRP Honeycomb Hollow Section Sandwich Panel ...................................... 46
Fig. 3-5 Bottom Surface Temperature Comparisons between the Measured and Predicted .................................................47
Fig. 3-6 Top Surface Temperature Comparisons between the Measured and Predicted .................................................48
Fig. 3-7 Load Tests Conducted at West Virginia University (Robinson 2001) ............................................................49
Fig. 3-8 Sketch of the GFRP Beam Finite Element Model .........................................................................................49
Fig. 3-9 Thermal Gradients of the C5C1 and C10C4 Cases .........................................................................................51
Fig. 3-10 Thermal Gradients of the G5C1 and G10C4 Cases .........................................................................................52
Fig. 3-11 Thermal Gradients of the C5S1 and C10S4 Cases .........................................................................................53
Fig. 3-12 Thermal Gradients of the G5S1 and G10S4 Cases .........................................................................................54
Fig. 3-13 Thermal Gradients of Bridges with Concrete Beam Cases ........................................................................55
Fig. 3-14 Thermal Gradients of Bridges with Steel Girder Cases ................................................................................56
Fig. 3-15 Temperature and Strain Distribution Diagrams .........................................................................................58
Fig. 3-16 Strain Distributions of the Selected Cases .................................................................................................58
Fig. 3-17 Strain Distributions of Concrete Beam Cases .............................................................................................59
Fig. 3-18 Strain Distributions of Steel Girder Cases .................................................................................................60
Fig. 3-19 Stress Comparisons between Concrete and GFRP Slabs for Concrete Beam Cases ................................61
Fig. 3-20 Stress Comparisons between Concrete and GFRP Slabs for Steel Girder Cases ........................................62
Fig. 4-1 GFRP Honeycomb Hollow Section Sandwich Panel Bridge ........................................................................69
Fig. 4-2 Sketch of the GFRP Panel Configuration .................................................................................................69
Fig. 4-3 HS-20 Truck Loading Positions ..................................................................................................................70
Fig. 4-4 HS-20 Truck Information ............................................................................................................................70
Fig. 4-5 Sub-structuring Modeling Procedures for the GFRP Panel Bridge under HS-20 Loads ..........................74
Fig. 4-6 Lateral Distribution Factor Results from ANSYS Predicted and Field Tests ...........................................75
Fig. 4-7 Measured Bridge and Ambient Temperatures from January 24 to July 13, 2004 .......................................76
Fig. 4-8 Positive and Negative Thermal Gradients Used in the Model .................................................................77
Fig. 4-9 Concrete Beam Bridge ..............................................................................................................................80
Fig. 4-10 Steel Girder Bridge .................................................................................................................................80
Fig. 4-11 Concrete and Steel Girders Geometry ................................................................. 80
Fig. 4-12 Temperature Gradients for Concrete and GFRP Slab Bridges with Concrete Beams .......... 82
Fig. 4-13 Temperature Gradients for Concrete and GFRP Slab Bridges with Steel Girders ............. 82
Fig. 4-14 Vertical Deflections of Steel Girder Bridge .................................................................. 85
Fig. 4-15 Vertical Deflections of Concrete Beam Bridge ............................................................ 85
Fig. 4-16 Steel Girder Bottom Surface Stresses of Steel Bridges ................................................. 86
Fig. 4-17 Concrete Beam Bottom Surface Stresses of Concrete Bridges ......................................... 86
Fig. 4-18 Steel Girder Top Surface Stresses of Steel Bridges ....................................................... 87
Fig. 4-19 Concrete Beam Top Surface Stresses of Concrete Bridges ............................................. 87
Fig. 4-20 Horizontal Movements of Steel Girder Bridges ............................................................ 88
Fig. 4-21 Horizontal Movements of Concrete Beam Bridges ....................................................... 88
Fig. 5-1 Schematic View of a Full Integral Abutment Bridge (IAB) .............................................. 93
Fig. 5-2 Caminada Bay Integral Abutment Bridge in the state of Louisiana .................................... 96
Fig. 5-3 Elevation View of the First 11 Spans of Caminada Bay Bridge ........................................... 96
Fig. 5-4 Plan and Side Views of Instrumentations Applied on the Slabs ........................................ 98
Fig. 5-5 Plan View of Instrumentations Applied on Bent1 ........................................................... 98
Fig. 5-6 Elevation View of Instrumentations Applied on Bent1 .................................................... 99
Fig. 5-7 Side View of Instrumentations Applied on Bent1 ........................................................... 99
Fig. 5-8 (a) Acquisition System on the Pole next to Bridge; (b) Acquisition System ....................... 99
Fig. 5-9 Normal Distribution Curve ......................................................................................... 100
Fig. 5-10 Normality Test of Span5 Strain Data at 08/11 17:00 .................................................... 101
Fig. 5-11 Normality Test of Span5 Temperature Data at 08/11 17:00 ............................................ 102
Fig. 5-12 Measured Environmental Conditions at the Weather Station ....................................... 104
Fig. 5-13 Hourly, Daily, and Monthly Varying Air Temperatures ................................................. 104
Fig. 5-14 Measured Hourly Varying Temperature of Bridge Slab ............................................... 106
Fig. 5-15 Bridge and Air Temperature during the Hottest Week from 08/19/11 to 08/25/11 .......... 107
Fig. 5-16 Bridge and Air Temperature during the Coldest Week from 12/29/11 to 01/06/12 ............. 107
Fig. 5-17 Measured Bridge Temperatures through Bridge Depth .................................................. 108
Fig. 5-18 Best Fitting of Air Temperature and Bridge Slab Average Temperature ......................... 109
Fig. 5-19 Measured Bent5 Top Surface Strain and Temperature .................................................. 110
Fig. 5-20 Measured Span5 Bottom Strain and Temperature ....................................................... 110
Fig. 5-21 Measured Strain at Span1 and Bent2 ............................................................................. 111
Fig. 5-22 Slab Strain Distribution w.r.t. Temperature Variations .................................................. 111
Fig. 5-23 Measured Bridge Temperature and Soil Deformation ................................................... 113
Fig. 5-24 Linear Fitting of Bridge Temperature Variations and Soil Deformation ......................... 113
Fig. 5-25 Measured Bridge Temperature and Bent1 Rotation .................................................... 114
Fig. 5-26 Measured Bridge Temperature and Bent11 Rotation ................................................... 114
Fig. 5-27 Measured Backfill Pressure at Middle Bent Location .................................................... 115
Fig. 5-28 Plan View of Pile Diagram .............................................................................................. 115
Fig. 5-29 Measured Pile Strains at B to E Cross Section .............................................................. 116
Fig. 5-30 Strain Components at B-B Section .................................................................................. 117
Fig. 5-31 Strain Components at C-C Section .................................................................................. 118
Fig. 5-32 Measured Exterior Pile Strain at Three Temperature Variations ..................................... 118
Fig. 6-1 Elevation View of the First 11 Span of the Caminada Bay Bridge ....................................... 127
Fig. 6-2 Plan View of the First 11 Span of the Caminada Bay Bridge ............................................. 127
Fig. 6-3 (a) Soil Layout from Boring Log 1; (b) Soil Layers for FEM Analysis ................................. 129
Fig. 6-4 Relationship between Wall Movement and Earth Pressure from NCHRP (1991) .............. 130
Fig. 6-5 F-D between Bent1 and Backfill at Three Elevations (a) Loose Sand; (b) Dense Sand ...... 130
Fig. 6-6 3D FEM of Caminada Bay Bridge Using ANSYS .............................................................. 131
Fig. 6-7 Slab Temperatures between Field Measurements and ANSYS Predictions ...................... 133
Fig. 6-8 Predicted Slab Temperature Gradients (a) Absolute Results (b) Normalized Results ........ 133
Fig. 6-9 Instrumentations Applied on Integral Abutment Bridge .................................................. 134
Fig. 6-10 Field Measured Temperatures on Slabs on (a) 08/30/11 and (b) 02/12/12 ........................................ 135
Fig. 6-11 Comparisons of Bent1 Displacements on (a) 08/30/11 and (b) 02/12/12 ........................................ 136
Fig. 6-12 Comparisons of Bent1 Rotations on (a) 08/30/11 and (b) 02/12/12 ........................................ 136
Fig. 6-13 Comparisons of Bent Top Surface Strains on (a) 08/30/11 and (b) 02/12/12 ........................................ 136
Fig. 6-14 Comparisons of Slab Bottom Surface Strains on (a) 08/30/11 and (b) 02/12/12 ........................................ 137
Fig. 6-15 Comparisons of Pile X-axis Bending Strains on (a) 08/30/11 and (b) 02/12/12 ........................................ 137
Fig. 6-16 Comparisons of Pile Y-axis Bending Strain on (a) 08/30/11 and (b) 02/12/12 ........................................ 137
Fig. 6-17 Bridge Responses under Different Support Condition Cases ......................................................... 140
Fig. 6-18 Bridge Responses under Different Backfills behind the Bent1 Cases .................................................... 141
Fig. 6-19 Bridge Reponses under Different Soils Surrounded the Piles Cases .................................................... 142
Fig. 6-20 Bridge Responses under Different Connection Details between Slabs and Bents Cases ............... 143
Fig. 7-1 Elevation View of the First 11 Spans of Caminada Bay Bridge .......................................................... 151
Fig. 7-2 3D FEM of Caminada Bay Bridge Using ANSYS ............................................................................. 151
Fig. 7-3 GFRP Hollow Section Sandwich Panel .............................................................................................. 152
Fig. 7-4 Represenative GFRP Slab and Equivalent Slab ................................................................................. 154
Fig. 7-5 Slab Vertical Displacement with X-direction Supports Fixed (a) GFRP (b) EQUIV ..................... 156
Fig. 7-6 Slab Vertical Displacement with Y-direction Supports Fixed (a) GFRP (b) EQUIV ..................... 156
Fig. 7-7 Measured Hourly Varying Temperature of Bridge Slab ...................................................................... 157
Fig. 7-8 Bridge and Air Temperature during the Hottest Week from 08/19/11 to 08/25/11 .................... 157
Fig. 7-9 Bent1 Displacements under Different Temperature Loadings and Support Conditions ................ 159
Fig. 7-10 Slab Strains under Different Temperature Loadings and Support Conditions ......................... 159
Fig. 7-11 Backfill Pressures under Different Temperature Loadings and Support Conditions ............... 160
Fig. 7-12 Pile Moment under Different Temperature Loadings and Support Conditions ....................... 160
Fig. 7-13 Bent Top Surface Stresses on (a) Free Support and (b) Fixed Support ..................................... 164
Fig. 7-14 Slab Bottom Surface Stresses on (a) Free Support and (b) Fixed Support .................................. 165
ABSTRACT

Expansion joints are often considered as one of the most vulnerable elements affecting the sustainability of traditional jointed bridges. Over the past several decades, a new type of integral abutment bridge (IAB) has been proposed, where the joints are eliminated at the abutments and/or along the length of the bridges. Although with wide acceptances, the IABs have not been largely applied in practice. Many arguments are unsettled and there are no national design guidelines currently. Among all, the thermal behavior is one of the most concerned issues, and that, to a large extent, limits the maximum length of IABs that can be constructed. Under this circumstance, a new type of fiber reinforced polymer (FRP) materials, with special material properties, are considered as an alternative to replace the traditional concrete and steel materials. However, the studies on the performances of both IABs and FRP bridges are not adequate. Therefore, an investigation on the thermal behaviors of IABs and FRP bridges is conducted. Then, an effort is made to analyze the responses by combining the FRPs with IABs, and to verify that such a configuration will help resolve the thermal issues of IABs.

For FRP bridges, (1) the temperature distributions of a GFRP panel are discussed based on a field monitoring program conducted at the state of Kansas; (2) the influencing factors on the temperature distributions are studied, including the material property, environmental condition, and section hollowness; (3) the thermal gradients of the FRP panel bridges are proposed referring to the AASHTO LRFD design code; and (4) the jointed bridges’ performances, after replacing traditional slabs by FRP panels, are numerically analyzed.

For IABs, (1) the thermal responses of the first full IAB in the state of Louisiana, Caminada Bay Bridge, are discussed based on a field monitoring program; (2) a parametric study is employed to analyze the effects of different parameters on the thermal performances, including the soil types, bent-pile connections, loading types, and support conditions; and (3) a numerical study is performed to verify the assumption that applying FRP panels on IABs will help resolve the thermal issues of IABs.
CHAPTER 1. INTRODUCTION

This dissertation is composed of eight chapters. Except for the first introduction and the last conclusion sections, all the other six chapters are written in a technical paper format which is approved by the Graduate School at LSU. All these six chapters are either being under review, or are to be submitted to the peer-reviewed journals for publications. The technical paper format is intended to facilitate and encourage publications of research results by graduate degree candidates. Thus, each chapter is independent, even though some reviewing and referencing information may be repeated for the completeness of these chapters. All chapters document the research work of the Ph. D. candidate under the major of advisor and committee members. This introductory chapter presents the general motivation of the study and the present and previous achievements related to this research topic. More detailed information can be found in the subsequent chapters.

1.1. Thermal Effects of Bridges

Temperature variations are significantly important on a bridge’s performance and are often divided into two components, i.e., the uniform and the gradient one. The former one, referring to the effective average temperature along a bridge cross section, will induce expansion and contraction movements; while the latter one, with varying temperatures from a bridge’s top to its bottom, will cause bending deformations. Any restraints to these thermal displacements and deformations will induce thermal stresses. For example, preventing those thermal movements, induced from the uniform temperature variations, will generate axial forces and compressive stresses; and the differential deformations through the cross sections, caused by either the nonlinear temperature gradients or by the different thermal expansion coefficients between materials, will develop internal self-equilibrating stresses. Thus, an accurate estimation of the bridge’s thermal responses is of great importance. It is especially fundamental in the selection of the joints and bearing systems (Arockiasamy et al. 2008; Roeder 2003; Ni et al. 2007), and also affects the designs of the presstressed concrete and steel members (Barr et al. 2005; Shoukry et al. 2009; Roberts-Wollman et al. 2002; Tong et al. 2001; Mahama et al. 2009).

The temperature field prediction methods for concrete and steel bridges have been well documented in the literature (Priestley 1978; Elbadry and Ghali 1983; Moorty and Roeder et al.
Generally, the temperature distribution at any time within a bridge is governed by a partial differential heat transfer equation, expressed as

$$\rho c \frac{dT}{dt} = k_x \frac{\partial^2 T}{\partial x^2} + k_y \frac{\partial^2 T}{\partial y^2} + k_z \frac{\partial^2 T}{\partial z^2}$$  \hspace{1cm} (1)

where, $\rho$ = the material density; $c$ = the material specific heat; $T$ = the bridge temperature; $t$ = the time; $k_x$, $k_y$, and $k_z$ = the material thermal conduction coefficients in the directions of $x$, $y$, and $z$, respectively; $x$, $y$, and $z$ = the coordinates in three directions. Under natural environmental conditions, the temperature distribution of the bridges is influenced by many factors; however, several major mechanisms are considered during the modeling and design stages for the purposes of simplifications, including the solar radiation, surface convection, surface radiation, and body conduction, expressed as,

$$\eta I_n - h_c (T_s - T_a) - \varepsilon \sigma (K_s^4 - K_a^4) + k_n \frac{dT}{dn} = 0$$  \hspace{1cm} (2)

where $\eta$ = the material solar absorptivity coefficient; $I_n$ = the solar input flux on bridge surfaces; $h_c$ = the convection coefficient between bridge surfaces and the adjacent air; $T_s$ = the bridge surface temperature; $T_a$ = the ambient temperature; $K_s$ = the absolute bridge surfaces temperature; $K_a$ = the absolute ambient temperature; $\varepsilon$ = the material emissivity coefficient; $\sigma$ = the Stefan-Boltzmann constant; and $k_n$ = the material thermal conduction coefficient.

For the boundary conditions of Eq. (2), three categories of parameters are needed, including the environment conditions (i.e., the solar flux and ambient temperature), material properties (i.e., the coefficients of thermal conductivity, solar absorptivity, and emissivity), and the coefficients of thermal convection related to the wind speed and material surface roughness. Specifically, the solar flux, ambient temperature, and wind speed can be conveniently obtained either from the local weather stations or from the field monitoring measurements. Also, they can be more efficiently and economically obtained through the numerical simulations. For example, the ambient temperature variations can be predicted by interpolating a sinusoidal curve with the given daily maximum and minimum temperatures; and the simulation of the solar input flux, though a little more complicated, can be calculated considering the solar input energy, time of the year, incident angles, effects of the atmosphere, air clearness conditions, etc. (Dilger et al. 1983; Duffie and Beckman 1980).
In order to relieve the thermal effects on the bridge structures, expansion joints are traditionally provided and designed to accommodate the thermal movements. After years of services, however, these joints are considered to be one of the most vulnerable elements affecting the sustainability of bridges. Firstly, the water or deicing chemicals, leaking through the joint gaps onto the underlying structures, lead to the steel deterioration and concrete spalling. Secondly, the expansion joints are often subjected to some cycling and devastating loadings; thus, they tend to be impaired and the damages could be more aggregated if the movements are obstructed. Lastly, the applications of joints would yield huge life-cycle maintenance expenditures from the beginning of the construction through the whole service life (Mistry 2005; and Thippeswamy et al. 2002). Under this circumstance, a new type of bridge configuration, integral abutment bridge (IAB), without joints at the abutments and/or along the length of the bridges, is proposed.

1.2. Study of Integral Abutment Bridges

IABs have been designed and constructed during the recent several decades with much success. The purpose of their applications is to eliminate the expansion joints and to resolve all the joints-induced problems. A full integral abutment bridge (IAB) refers to a single or multi-span bridge which has its superstructure cast monolithically with the substructure. In such a configuration, the horizontal movements from the superstructure are transferred to the substructure and further accommodated by the complicated soil-structure interaction behaviors. Both the field monitoring studies and numerical investigations have been conducted on the IABs by the Department of Transportations (DOTs) and research institutes around US. However, there are still many unsettled arguments so that no national design guidelines exist and the current constructions are primarily relying on empirical practice.

Through field monitoring studies, some of the design and construction assumptions are studied and justified, such as: (a) the maximum allowable design criteria (e.g., total and individual bridge span lengths and skews); (b) the structure design parameters (e.g., types and orientations of the pile, abutment, and wingwall); (c) the soil-structural interaction behaviors (e.g., between the soil-pile, abutment-backfill, and approach slab-backfill); (d) the joint connection detail effects (e.g., at the interfacial locations between the abutment-deck-girder, abutment-pile cap, approach slab-abutment, and intermediate pier-girder); (e) the stress relief
mechanisms (e.g., the diameters, depths, and filling materials of the pre-sized holes surrounding the pile, and the compacting degree of the backfill materials behind the abutment); and (g) the long term effects (e.g., thermal, shrinkage, creep, and steel relaxation) (Dunker and Liu 2007; Arockiasamy et al. 2004; Huang et al. (2004); Abendroth and Greimann 2005; Frosch et al. 2006; Hassiotis et al. 2006; Laman et al. 2006; Brena et al. 2007; Hoppe and Bagnall 2008; Ooi et al. 2010; Davids et al. 2010).

Through numerical modeling studies, the structural and geotechnical parameters are varied in the finite element models to study the behaviors of IABs under different conditions by some state Department of Transportations (DOTs) and research institutes (Faraji, et al. 1999; Arockiasamy et al. 2004; Steinberg et al. 2004; Dicleli 2005; Fennema et al. 2005; Khodair and Hassiotis 2005; Dicleli and Erhan 2008; Pugasap, et al. 2009; Ooi et al. 2010). For example, Huang et al. (2008) examined the effects of the structural configurations, i.e., the hinged and fixed connections at the abutment-pile cap, weak and strong axes bending of the steel and concrete piles, etc.; Civjan et al. (2007) discussed the soil effects both from the compacting degrees of the backfills behind the abutments and the soil restraints surrounding the upper part of the piles; Thippeswamy et al. (2002) compared the responses of the primary and secondary loading effects, including the dead load, creep of material, live load, thermal gradient, uniform temperature change, shrinkage, differential settlement, and earth pressure.

Based on the findings of these investigations, together with the field monitoring and numerical studies conducted by the authors of this dissertation on a full IAB located in the state of Louisiana, Caminada Bay Bridge, a dilemma is found during the designs of IABs. It is difficult to both release the restraints to the thermal movements at the superstructure and, at the same time, reduce the thermal forces on the substructure. This problem can be attributed to the relatively larger thermal expansion coefficients but smaller tensile strength capacities of the concrete materials. Thus, the benefits of IABs may not be extended and adapted to those bridges with longer spans considering the correspondingly larger thermal deformations and greater thermal forces. In this case, fiber reinforced polymer (FRP) materials, with special thermal properties and higher strength capacities are studied as the alternatives to replace the concrete superstructure for IABs.
1.3. Study of FRP Panel Bridges

FRP materials have been widely applied in the military industry and aerospace field for a long time. In bridge engineering, however, they just begin to be adopted in practice in the recent decades. Among all the applications, the glass FRP (GFRP) panel, with light weight, high strength, good corrosion resistance, and long term durability, is considered to be one of the most prosperous alternatives and have already been applied to the bridge replacements, retrofits, and rehabilitations.

The behaviors of the GFRP panels in bridge engineering are investigated by researchers, such as in the aspects of the static performances (Camata and Shing 2005; Zhang and Cai 2007; Turner et al. 2004), and dynamic performances (Zhang et al. 2006; Aluri et al. 2005). Also, other tentative studies are performed on their thermal behaviors in terms of the temperature distributions, thermal deformations, strains, and stresses. The thermal properties of the GFRP panels, i.e., the solar absorption ability, convection coefficient, conduction coefficient, and expansion coefficient, are all different compared to that of the concrete and steel materials. Thus, the induced thermal responses are expected to be distinct (Laosiriphong et al. 2006; Liu et al. 2008; Reising et al. 2004). In addition, among all the studies of FRP bridges, one of the most difficult aspects is to accurately predict the material properties from the micro and macro perspective of views, and also efficiently develop the finite element models for those FRP structures with complex geometry configurations. Thus, several simplified modeling methods are proposed including the one-layer model, three-layer model, and simplified I-beam model (Cai et al. 2009; Davalos et al. 2001; Morcous et al. 2010). Even though with some differences between each other, they are basically sharing a similar concept, namely, to homogenize the complicated properties of the lamina, laminate, and panel into a correspondingly equivalent one.

1.4. Overview of the Dissertation

The motivation of this study is firstly to conduct a more in-depth investigation on the thermal behaviors of IABs and FRP panel bridges, respectively, using the field monitoring and numerical simulation methods. Then, a tentative study is performed to analyze the behaviors of IABs after replacing the concrete slabs with FRP panels. The results will be used to verify the assumption that adopting FRP panels can help resolve the dilemma that introduced above during the designs of IABs. Following this idea, the contents of each chapter are organized as follows.
In Chapter 2, the temperature field of a GFRP honeycomb hollow section sandwich panel is investigated. Firstly, based on a field monitoring program conducted by Kansas DOT, the temperature distribution pattern of the top and bottom surfaces of this panel is discussed. Then, a transient state thermal field prediction model, together with the thermal properties and environmental conditions, is proposed and validated by comparing the modeling results with the field measurements. Finally, a parametric study is conducted to investigate the effects of the section hollowness, thermal property parameters, and environmental conditions on the temperature distribution of the GFRP panels.

In Chapter 3, a comparison of the thermal behaviors is performed between the bridges with different material types and geometrical configurations, aiming to improve and extend the adaptability of the current AASHTO LRFD (2007) temperature design criteria on the glass FRP (GFRP) composite bridges. The mechanical and physical material properties of the FRP panels are predicted using the micro-macro mechanics theory and verified by a field monitoring program and an experimental test. Then, the temperature distribution patterns of the GFRP panel bridges are proposed by comparing them with that of the concrete and steel bridges available in the AASHTO LRFD (2007) design code. Finally, the thermal strains and stresses induced from the temperature gradients are examined.

In Chapter 4, a parametric study is employed to study the behaviors of bridges after replacing the concrete slabs with GFRP panels based on one field study program conducted at the state of Kansas. First, a numerical finite element model, using a sub-structuring modeling method, is proposed with the help of the commercial software ANSYS 11.0 and verified by comparing the predicted live load distribution factors with that of the live load tests. Second, with this model, the bridge’s thermal behaviors are discussed using the measured bridge temperatures from January 24 to July 13 in 2004. Finally, a parametric study is conducted to compare the behaviors of two general slab replacement cases, i.e., replacing concrete slabs with GFRP panels for both a concrete beam and a steel girder bridges, respectively. The induced deformations and stresses under the uniform temperature variations, nonlinear thermal gradients, dead loads, and HS-20 live loads are investigated.

In Chapter 5, it presents the field measurements obtained from a monitoring program on the first full integral abutment bridge in the state of Louisiana, Caminada Bay Bridge on the soft
soil condition, over a year since August, 2011. A total of 81 instrumentations are applied on the bridge with the purpose of investigating the bridge responses due to the daily and seasonal temperature variations. The results will also serve to provide references for future constructions of IABs on such soil and environmental conditions in the state of Louisiana. The discussions in this chapter are primarily emphasized on the observed bridge and environmental temperature distributions, abutment rotations and displacements, slab positive and negative bending strains, pile strains, and backfill pressures.

In Chapter 6, a numerical study is performed to investigate the effects of different structural and geotechnical parameters on the thermal responses of IABs. A 3D finite element model is firstly developed using the commercial software ANSYS 11.0 considering the soil-structure interaction behaviors. Then, the model is verified by comparing the simulated thermal responses with that of the field measurements at the two representative hot and cold days. Finally, the parameters, including the support conditions, soil types behind abutment and surrounding the piles, and joint connections at the interfacial locations between the pile and bent are varied in the numerical models and the corresponding responses under the uniform and gradient temperatures are discussed.

In Chapter 7, a numerical study is conducted to study the performances of one as-built IAB, i.e., Caminada Bay Bridge designed by the LADOTD, after replacing the concrete slabs with the popular FRP panels in the market, i.e., GFRP honeycomb hollow section sandwich panel manufactured by Kansas Structural Composite, KSCI. First, a homogenization and stiffness-equivalent method is employed to predict the equivalent elastic and thermal properties of the GFRP panel. Then, the behaviors of IABs after replacing concrete slabs by GFRP panels under the uniform and gradient temperatures specified by AASHTO LRFD (2007) are discussed.

In Chapter 8, it concludes all the research results of this dissertation. Firstly, for the study of the FRP panel bridges, it summaries (1) the temperature variations patterns of the GFRP panels, (2) the temperature gradient distributions of the FRP panel bridges for designs, and (3) the performance of concrete and steel bridges before and after slab replacements with FRP panels. Secondly, for the study of the IABs, it summaries (1) the structural and geotechnical responses of the Caminada Bay IAB due to the temperature variations; and (2) the behaviors of more general IABs under different soil types, connection behaviors, support conditions, and
loading types. Thirdly, for the study of the IABs with FRP panels, it verifies the assumption that the innovative structural configuration, by applying FRP panels on IABs, will resolve the thermal issues of IABs. Finally, in the last section of this chapter, some of the future study aspects are recommended both in the design and research fields.

1.5. References


CHAPTER 2. TEMPERATURE DISTRIBUTION BEHAVIORS OF GFRP HONEYCOMB HOLLOW SECTION SANDWICH PANELS

2.1 Introduction

Composite materials have been initially developed and applied to the military aircrafts and aerospace equipment since World War II. In bridge engineering, the early design and construction of composite bridges were also intended for military services, such as for the bridge quick erections and military force deployments. During the past several decades, with the gradual decrease of the material costs and the quick development of the manufacture techniques, composite materials have begun to be applied in the civilian areas. Nowadays, FRP composite panel bridges, with superior benefits of high strength, light weight, quick installation time, good corrosion resistance, and long term durability, have been considered as one of the alternatives for the replacements and retrofits of the structurally deficient or functionally obsolete bridges. A state-of-the-art survey and the present and future utilizations of FRP composites in civil engineering have been well reviewed in the literature (Foster et al. 2000; Hollaway 2010; Bakis et al. 2002).

Even though having these benefits, the FRP structures have not been widely applied in practice. The reasons, partly due to the concerns on the initial material costs, are primarily due to the lack of national design guidelines, and that in turn are attributed to the uncertainty on the performance of FRP bridges with their the complex material and structural properties. Unlike the traditional concrete and steel, composite materials are normally heterogeneous and non-isotropic; and their properties are largely determined by the constituents, i.e., fiber types, volume percentages, fiber orientations, resin types, manufacture methods, and bonding materials. This special characteristic of the material property may help engineers design composite structures for specific demands, e.g., the high strength FRP laminates for structure rehabilitations or light weight FRP decks for slab replacements; however, these designed FRP structures may, at the same time, show some negative performances on other aspects. The thermal response of the FRP panels discussed in this study is one of the recent concerns.

The thermal properties of FRPs, such as the thermal expansion and conduction coefficients, are different from that of the steel and concrete; and they are also different in the directions parallel and perpendicular to fibers (Tipirneni 2008). Thus, when applied to bridges or assembled
with other structural elements, these FRP structures will produce complicated thermal responses that may not satisfy the current strength or service limits and may also cause damages on other structure members. For example, as indicated by the Federal Highway Administration (FHWA) (http://www.fhwa.dot.gov/bridge/frp/deckprac.cfm, 2011), when exposed to the direct sunlight, the FRPs may be heated rapidly and the large temperature differences are produced between the top and bottom deck surfaces. These large thermal gradients, on one hand, will produce the hogging and sagging actions affecting the performance of the bearings, anchorage details, and the panel movements; on the other hand, they also will induce thermal stresses causing wearing surface cracks and the delamination or debonding failures.

The research on the thermal behaviors is no longer a new topic for traditional concrete and steel bridges, while the study on the thermal behaviors of FRP composite bridges in civil engineering is not adequate. Oghumu (2005) developed a full 3D finite element model to investigate the local stresses and delamination problems at the interfaces between the facial and core of a GFRP sandwich panel under the uniform and linear temperature gradients; Laosiriphong et al. (2006) studied the thermal performance of a FRP deck through the lab tests and correlated the thermal responses with theoretical results; Liu et al. (2008) observed the first built No-Name Creek FRP composite bridge in US under the temperature loadings and discussed the induced deflections; and Reising et al. (2004) compared four different FRP composite panels under the same environmental conditions and investigated the temperature induced delamination failures, large thermal gradients, and panel movements. Besides these studies, the experimental tests or numerical studies, however, are nearly, if any, having been conducted to investigate the temperature distribution patterns for those recently adopted FRP panels in bridge engineering. An accurate prediction of temperature distribution patterns is always significantly important to understand the thermal performances of bridges, and also crucial in the further analysis of the thermal deformations and forces.

In this sense, a field monitoring program, conducted by Kansas Department of Transportation (DOT) on a GFRP honeycomb hollow section sandwich panel bridge, located at Crawford County, KS, provides the precious information to analyze its temperature distributions. The studied GFRP deck was manufactured by Kansas Structural Composite, Inc. (KSCI) through the hand lay-up technique, where temperature sensors were mounted on the top and bottom
surfaces of the panels, and the temperatures of the panels and the real time ambient were recorded every two hours from December 2002 to July 2004. Therefore, this paper firstly demonstrates and discusses the temperature distributions of the GFRP panel based on the field monitoring results. Then, a transient state thermal field finite element analysis is conducted using the commercial software ANSYS 11.0 to predict the thermal gradients along the panel depth, where the proposed temperature modeling method, the predicted GFRP thermal properties, and the adopted environmental parameters are validated through comparing the results with the field monitoring measurements. Finally, a detailed parametric discussion is conducted to investigate the effects of the entrapped air, section hollowness, and the thermal and environmental parameters on the thermal gradient distribution of the GFRP panel.

2.2 Heat Transfer Mechanism

Temperature field modeling theories and analytical solution methods for concrete and steel bridges have been well documented in the literature (Priestley 1978; Elbadry and Ghali 1983; Moorty and Roeder et al. 1992; Mirambell and Aguado 1990), and the important parts are briefly summarized in this section for the convenience of discussions. More importantly, these heat transfer theories will be examined in the following field monitoring discussions and utilized in the numerical modeling studies.

2.2.1 Heat Transfer Equation

The temperature distribution at any time within a bridge is governed by a partial differential heat transfer equation as:

$$\rho c \frac{\partial T}{\partial t} = k_x \frac{\partial^2 T}{\partial x^2} + k_y \frac{\partial^2 T}{\partial y^2} + k_z \frac{\partial^2 T}{\partial z^2}$$

(1)

where, \(\rho\) = the material density; \(c\) = the material specific heat; \(T\) = the bridge temperature; \(t\) = the time; \(k_x\), \(k_y\), and \(k_z\) = the material thermal conduction coefficients in the directions of \(x\), \(y\), and \(z\), respectively; \(x\), \(y\), and \(z\) = the coordinates in three directions. For a bridge under natural environmental conditions, the 3D temperature transferring behavior, expressed as Eq. (1), is often simplified into a 2D one where the temperature variations along the direction of the bridge length are observed too small to be neglected. The bridge temperature differences in the transverse direction, noteworthy for those bridges with large thermal inertia, such as the large box girder bridges, can be reasonably disregarded for the GFRP panel studied in this project due
to its small thermal inertia. Then, the 2D thermal behavior is further simplified into a 1D one in the numerical modeling of the current GFRP panel.

### 2.2.2 Boundary Conditions

A bridge thermal field under natural environmental conditions is influenced by many factors, e.g., solar direct radiation, solar diffuse radiation, bridge surface re-radiation, atmospheric counter radiation, ground radiation, structure mutual radiation, convection between bridge surfaces and the ambient, wind, rain, snow, etc. Modeling all of these factors are infeasible and unnecessary since most of them make negligible or short-term effects and will not be considered as critical ones in design. In this study, therefore, three major mechanisms are considered, including the conduction heat transfer behavior through the bridge body due to temperature differences governed by the Fourier’s equation, the convection heat transfer behavior between the surfaces of the structures and the adjacent air described by the Newton’s law of cooling, and the material radiation heat transfer behavior by emitting and receiving the electromagnetic energy defined by the Stefan-Boltzmann Law. Thus, the equilibrium equation for the solar radiation, surface convection, surface radiation, and body conduction boundaries can be respectively expressed as:

\[
\eta I_n - h_c (T_s - T_a) - \epsilon \sigma (K_s^4 - K_a^4) + k_n \frac{\partial T}{\partial n} = 0
\]  

(2)

where \(\eta\) = the material solar absorptivity coefficient; \(I_n\) = the solar input flux on bridge surfaces; \(h_c\) = the convection coefficient between bridge surfaces and the adjacent air; \(T_s\) = the bridge surface temperature; \(T_a\) = the ambient temperature; \(K_s\) = the absolute bridge surfaces temperature; \(K_a\) = the absolute ambient temperature; \(\epsilon\) = the material emissivity coefficient; \(\sigma\) = the Stefan-Boltzmann constant; and \(k_n\) = the material thermal conduction coefficient.

### 2.2.3 Environmental and Thermal Parameters

Considering the thermal boundary conditions of Eq. (2), three categories of parameters are needed as input in modeling, including the environment conditions (i.e., solar flux and ambient temperature), the material properties (i.e., coefficients of thermal conductivity, solar absorptivity, and emissivity), and the coefficients of thermal convection related to the wind speed and material surface roughness. In the numerical modeling, the solar flux, ambient temperature, and wind
speed can be conveniently obtained either from the local weather stations or from the field monitoring measurements. If the weather stations are not available near the bridge sites or the required hourly-varying data is not obtained, the environmental conditions could be more efficiently simulated through the numerical methods. For example, the ambient temperature variations are often predicted by interpolating a sinusoidal curve using the available daily maximum and minimum temperatures; and the simulation of the solar input flux, though a little more complicated, is calculated considering the solar input energy, time of the year, incident angles, effects of the atmosphere, air clearness conditions, etc. (Dilger et al. 1983; Duffie and Beckman 1980).

Regarding to the material properties, unlike traditional concrete and steel in which plenty of researches have been conducted and the empirical parameters of conduction, convection, solar absorptivity and emissivity are available in the design manuals of civil structures, the material properties of FRPs are case-dependent and largely determined by the constituents. Methods have been provided to predict the composite material properties, and an analytical method using the knowledge of micro-macro mechanics, commonly utilized in a routine design, is adopted in this study. Generally, the FRP material is in laminated configurations composed of a few layers with different fiber orientations and volume percentages, where each of these layers is called a lamina. The micro-mechanical analysis is conducted to develop the material properties for each lamina, and the whole laminate properties are obtained by assembling the properties of each lamina through the macro-mechanical analysis thereafter. This method can provide reasonable results for the uniformly arranged materials like unidirectional fiber components (UNC) since material properties are dominant in the direction parallel and perpendicular to the fibers. For some others with randomly oriented fibers, however, the analytical results may produce some errors but still be within a certain tolerance for engineering applications. Theories and equations adopted for deducing the thermal properties of the GFRP panels are referred to the literature (Composite Materials Handbook 2001; McCartney and Kelly 2007).

2.3 Field Monitoring Study

2.3.1 Project Description

The studied bridge was originally an asphalt-on-steel deck supported by fourteen W21×68 I-beam stringers, and the deck was replaced by five GFRP honeycomb hollow section sandwich
panels laying perpendicular to the girders, shown in Figure 2-1. Kansas DOT measured the temperatures at top and bottom surfaces of the panel and the ambient every two hours from December 2002 to July 2004 (Meggers 2006).

![Fig. 2-1 GFRP Honeycomb Hollow Section Sandwich Panel Bridge](image)

**2.3.2 Monitoring Results**

The measured temperature data, from January 24 to July 13 in 2004, plotted in Figure 2-2, is selected and used throughout the following discussions. Figure 2-3 and Figure 2-4 show the detailed hourly-varying temperatures during the warmest and coldest week, respectively. In this section, the temperature distribution is observed and analyzed with the purpose of clarifying the following uncertainties, including (1) the temperature variation patterns of the GFRP panel, (2) the differences between the measurements and the available AASHTO LRFD (2007) temperature design specifications for concrete decks, and (3) the effects of the environmental and material parameters on temperature variations. Since the wind speed, also one of another important influencing factors, has not been measured in the field, then, the data from the nearest weather station, Konza Prairie Biological Station, located at Manhattan, KS, with the latitude and longitude of 39.1027N, 96.6098W ([http://www.ncdc.noaa.gov](http://www.ncdc.noaa.gov), 2012), is obtained for reference. Based on the observation, some of the findings can be summarized as follows:
Fig. 2-2 Measured Temperature from January 24 to July 13, 2004

Fig. 2-3 Measured Temperature from June 21 to June 27, 2004
Fig. 2-4 Measured Temperature from Feb.7 to Feb.14, 2004

Fig. 2-5 Environmental Conditions from Field Measurements and Weather Station
Fig. 2-6 Temperature Linear Fitting Results of the Bottom Surface and the Air

Fig. 2-7 Temperature Linear Fitting Results of the Top Surface and the Air
Figure 2-2 shows the temperature variations at the panel surfaces from January 24 to July 13 in 2004. It can be observed that the measured temperatures are more significantly fluctuated at the top surface as opposed to the less varying ones on the bottom surface. This phenomenon indicates that the GFRP top surface temperature, evidently being higher than that of the air, is decisively affected by the solar radiation together with the material energy absorption ability; while the GFRP bottom surface temperature, almost being consistent with that of the air, is basically affected by the air convection below the panel. At the same time, the high and varying top surface heat energy cannot be easily transferred through the panel depth, so that it only makes negligible effects on the temperatures at the bottom surface, and this behavior may be attributed to the lower GFRP thermal conduction properties and the hollow section configurations.

The night sky radiation behavior, for the case when the bridge temperatures are lower than that of the ambient, is also observed in Figure 2-2. Specifically, during the night, when the bridge surface faces the night sky, it loses heat by radiation to the sky and gains heat from the surrounding air by convection. If the surface is a good radiation emitter or the convection is weak, it will tend to radiate more heat to the sky than it gains from the air, and the net result is the surface temperature dropping below to that of the air. Since the night sky radiation behavior often induces negative thermal gradients, thus the frequent occurrence of this behavior on GFRP panels in this project may indicate that the negative thermal gradients are common for GFRP panel bridges and the attentions should be given in design.

The hourly-varying sinusoidal temperature variations are clearly observed from the plots of the warmest and coldest week temperature distributions in Figure 2-3 and Figure 2-4. Therefore, the sinusoidal fitting algorithm discussed in the previous analytical modeling section is proven to be reasonable even though the lagging effect is not evident for this GFRP panel due to its shallower configurations and the smaller thermal inertia.

In addition, the measured maximum positive and negative temperature differences between the top and bottom surfaces during the warmest and coldest week are 28°C (51°F) and 7°C (12°F), respectively. Considering the design temperature stipulated for concrete slabs in AASHTO LRFD (2007), it specifies a 25°C (46°F) for positive gradients, and multiplied the positive gradients by -0.3 for plain concrete decks and -0.2 for asphalt overlaid decks, respectively, to obtain negative gradients. It can be calculated that the temperature differences of
the current GFRP panel, though not the worst case yet, are already extending the range that is specified for concrete slabs. Thus the available temperature design guidelines are no longer effective for GFRP slabs.

(3) Figure 2-5 shows the measured environmental conditions at both the bridge site and the weather station. The environments at the weather station can approximately represent that at the bridge site since the measured air temperatures are almost the same. The measured wind speed at the weather station is generally lower than 5m/s (16.4ft/s). Under this wind speed, the thermal coefficient of convection for a concrete deck will be approximately less than 20 w/m²°C (3.5Btu/hrft²°F) (Elbadry and Ghali 1983). In this sense, even though it is still unclear about the actual convection coefficient for a GFRP panel, yet considering the fact of almost consistent temperatures between the deck bottom surface and the air, it can be estimated that evident convection behaviors may happen under current wind speeds and the value of 20 w/m²°C (3.5Btu/hrft²°F) will be a good reference to be used in the following numerical models as the convection coefficient of the GFRP panel.

(4) The relationship between the bridge surface and the ambient temperatures are linearly fitted. Obviously, the bottom surface shows almost the same temperature as that of air, shown in Figure 2-6, while it is not the case for the top surface where a discrete pattern exists, shown in Figure 2-7. Therefore, for this specific GFRP panel, it is reasonable to use the ambient temperature to represent the bottom surface temperature. For the top surface temperature, however, it can be further divided into two scenarios. During the night without external solar radiation, the bridge surface temperature tends to approach the ambient temperature, and the linear trend is still valid; while during the daytime with solar radiation, the linear trend no longer exists.

2.4 Numerical Modeling Study

The field monitoring results discussed above only provide a qualitative view, while a comprehensive understanding of the GFRP panel temperature distributions is still unclear. On one hand, the measured data only includes the temperatures of the top surface, bottom surface, and ambient. No information can be obtained for thermal gradients through the slab depth; however, the thermal gradient is often of more importance. For example, the self-equilibrating thermal stress induced from the nonlinear thermal gradients is considered to be one of the major reasons causing the wearing surface cracks and delamination failures. On the other hand, the
influences of the thermal and environmental parameters on the temperature distribution patterns are only qualitatively observed from the monitoring results, and a quantitative analysis is still required to obtain a more accurate understanding of the effects from these parameters. Therefore, a numerical modeling method is proposed, verified, and discussed in this section.

2.4.1 GFRP Panel Thermal Properties Prediction

The GFRP panel material properties are predicted based on the knowledge of the micro-macro mechanics. The panel that will be studied is plotted in Figure 2-8. Basically, the sandwich panel is made of the E-glass fiber and polyester resin and consisted of three parts including two facial laminates at the top and bottom parts and one core in the middle. The special configuration of the sinusoidal core is designed with the flute half-wave length 10.16 cm (4in) and flute width 5.08 cm (2in).

The layers that constitute the facial laminate includes the chopped strand mat (ChSM), continuous strand mat (CSM), bidirectional stitched fabrics, and unidirectional (UNC) layers. Detailed information in terms of the fabrication techniques, geometry descriptions, and constituent layouts of the constituents are referred to the studies in the literature (Cai et al. 2009; Qiao and Wang 2005), and the thermal properties are predicted here. From the micro-mechanical perspective of view, the ChSM and CSM layers, made of continuous randomly oriented fibers having the same properties in all directions, are modeled as isotropic layers. The UNC layers,
with different material properties in the direction parallel and perpendicular to the fibers, are assumed orthotropic. Therefore, the thermal properties for the facial and core laminates are derived and shown in Table 2-1.

**Table 2-1 Constituents and Laminates Material Properties**

<table>
<thead>
<tr>
<th>Material</th>
<th>Density Units</th>
<th>Conductivity Units</th>
<th>Specific Heat Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>E-glass fiber</td>
<td>2.55 (0.092)</td>
<td>1.3 (0.75)</td>
<td>840 (0.2)</td>
</tr>
<tr>
<td>Polyester resin</td>
<td>1.14 (0.041)</td>
<td>0.2 (0.12)</td>
<td>1686 (0.4)</td>
</tr>
<tr>
<td>Facial laminate</td>
<td>1.59 (0.057)</td>
<td>0.456/0.365/0.333 (0.26/0.21/0.19)</td>
<td>1415 (0.34)</td>
</tr>
<tr>
<td>Core Mat</td>
<td>1.47 (0.053)</td>
<td>0.308 (0.18)</td>
<td>1486 (0.36)</td>
</tr>
</tbody>
</table>

*Note: The densities of the fiber and matrix are given by the manufacture and the conductivity and specific heat are referred to the online material property database MATWEB (http://www.matweb.com/) and the literature (Bai et al. 2008).*

### 2.4.2 Modeling Method

The thermal transfer mechanisms introduced in the previous section are simulated with the help of the commercial software ANSYS 11.0. The boundary conditions and temperature distributions are time-varying. Thus, a transient state thermal field analysis for the GFRP panel using the solid thermal elements, shown in Figure 2-9, is conducted. In the model, the convection boundaries are defined by the surface film coefficients and bulk temperatures, and the radiation boundaries are applied to the surface effect elements attached on the surface of solid elements. The entrapped air effects and inner surfaces mutual radiation effects within the hollow section are neglected at this stage and will be discussed in the following parametric study section. The numerical modeling duration is selected for the warmest week from 02:00 in 6/21/2004 to 24:00 in 6/27/2004 with a two-hour interval for the convenience of comparisons and verifications. The environmental ambient temperature is directly input from the field measurements, and the solar flux is calculated based on the available algorithms introduced above. The initial bridge temperature is assumed as the average value of the bridge maximum and minimum temperatures. The errors induced from the initial temperature assumptions will be offset by running programs three times under the same environmental conditions. All the parameters used in the finite element model are shown in Table 2-2.
2.4.3 Comparison of Results

The modeled temperature results of the top and bottom surfaces from June 21 to June 27 in 2004 are plotted in Figure 2-10 and Figure 2-11. It is seen through comparisons that the results from the measured and predicted are matching well. Thus, the modeling method together with the corresponding thermal properties and environmental parameters is verified. Figure 2-12 shows the results of the maximum positive and negative gradients appearing at 12:00 and 20:00 on June 23, and the plotted data has been normalized after subtracting the minimum temperature values along the panel depth. It can be observed that the nonlinear distribution pattern, although not very evident, is produced along the depth of the slab.
Fig. 2-10 Comparison of Top Surface Temperature between Modeled and Measured Results

Fig. 2-11 Comparison of Bottom Surface Temperature between Modeled and Measured Results
2.5 Parametric Study

With the proposed modeling method, the effects of section hollowness, environmental conditions, and material thermal properties on the GFRP panel temperature distributions are further investigated by a parametric study. All the parameters adopted in the above numerical model are used again, and the corresponding results, without the considerations of section hollowness effects, are taken as the base case for comparisons throughout the following discussions. Differently, considering the fact that the 12.7cm (5in) GFRP panel used in the project does not show obvious nonlinear thermal gradients, a more popular and deeper 25.4cm (10in) GFRP panel with the same material properties is selected and analyzed. Therefore, the modeled maximum positive and negative thermal gradients that occurred at 12:00 and 22:00 on June 23, 2004, respectively, are illustrated and discussed. Specifically, the thermal gradients are plotted for both the absolute and the normalized data where the normalized data refers to the results after subtracting the minimum values from the absolute values along the panel depth, i.e., by setting the minimum value as zero.

Fig. 2-12 Modeled Thermal gradients on June 23, 2004
2.5.1 Air Convection

During the daytime, the top surface of the panel is heated by the solar radiation, and the high energy will continue conducting through the panel depth and heating both the structure elements and the air inside the hollow section. During the night, with the external heating decrease and surface cooling, the heat is reversely transferred from the inner to the outer of the panel. In this way, the induced temperature differences between the inner air and the surrounding structural surfaces in the hollow section will develop an additional thermal convection system. The effects of this behavior on thermal gradients are simulated using the finite element method with ANSYS 11.0 by varying the thermal convection coefficients $h_c$ between the inner surfaces and the inner air temperatures within the hollow section, where the air temperature is assumed to be the same as that of the measured ambient.

Figure 2-13 shows the absolute temperature distribution due to the entrapped air convection effects during the daytime with different $h_c$ thermal convection coefficients. It can be observed that the inner convection behavior has more evident effects on the temperatures at the panel’s top surface than that at the bottom. When the temperature of the top surface is higher than that of the air, with the increase of the convection behaviors, the temperature values tend to be decreased; while at the bottom surface, the structural temperature tends to be increased. In addition, the effects of the inner convection behavior on the thermal gradients during the daytime are also evident, shown in Figure 2-14. With the increase of the $h_c$ convection coefficients, the nonlinear thermal gradients are obviously linearized especially for the locations away from the top surface, even though the top surface’s temperature is still largely determined by the solar radiation.

The similar phenomenon can also be observed for negative thermal gradients during the night. As is shown in Figure 2-15, when the top surface’s temperature is consistent with that of the air during the night, the convection behavior almost disappears, while the higher bottom surface’s temperature still tends to approach the air temperature with the increase of convection behaviors. Thus, the final maximum and minimum negative thermal gradients, shown in Figure 2-16, would be happening at the conditions with the smallest and largest $h_c$ thermal convection coefficients, respectively. Therefore, it can be concluded that the inner air convection behavior tends to reduce the nonlinear gradient patterns and to make the temperatures more uniformly
distributed; and if ignoring the inner air convection behavior in the modeling, the induced thermal gradients are actually larger.

**Fig. 2-13** Absolute Thermal Gradients under Convection Effects at 12:00, June 23

**Fig. 2-14** Normalized Thermal Gradients under Convection Effects at 12:00, June 23
Fig. 2-15 Absolute Thermal Gradients under Convection Effects at 22:00, June 23

Fig. 2-16 Normalized Thermal Gradients under Convection Effects at 22:00, June 23

2.5.2 Mutual Radiation

Besides the convection behaviors, the temperature differences at the surrounding structural surfaces inside the hollow section would also generate a mutual radiation thermal transfer
mechanism. The effects of this mechanism on thermal gradients can be simulated by ANSYS 11.0 using the coded AUX12 Radiation Matrix Method where the form factors are defined to account for the relative positions between any two radiated surfaces. The material surface emissivity properties, referring to the ratio of the radiation emitted by a face to the radiation emitted by a black body at the same temperature, are varying in the modeling.

Figure 2-17 shows the thermal gradients due to the inner surface mutual radiation at noon. With the increase of the emissivity coefficients labeled as “emis” in the figure, the top surface temperatures are negligibly affected, while the bottom surface temperatures are increased apparently. Also, the nonlinear temperature distribution, shown in Figure 2-18, tends to be smoothed out, and the highest mutual radiation coefficient leads to almost a linear temperature gradient distribution pattern. Likewise, during the night, with the increase of the emissivity, the bottom surface temperatures, shown in Figure 2-19, are also increased a little bit while the nonlinear temperature gradients, shown in Figure 2-20, are significantly softened. In this sense, it can be also concluded that with the consideration of inner surface mutual radiation, the nonlinear thermal gradients tend to be linearized.

![Diagram showing thermal gradient effects under radiation]

Fig. 2-17 Absolute Thermal Gradients under Radiation Effects at 12:00, June 23
Fig. 2-18 Normalized Thermal Gradients under Radiation Effects at 12:00, June 23

Fig. 2-19 Absolute Thermal Gradients under Radiation Effects on 22:00, June 23
2.5.3 Environmental and Material Properties

The effects of the material properties and environmental conditions on GFRP panel thermal gradients are discussed by comparing the thermal responses after artificially increasing and decreasing all the parameters by 25%. The temperature changes due to the varying parameters at the top surface, middle section, bottom surface, and the temperature differences between the maximum and minimum temperatures through the depths at 12:00 on June 23, 2004 are listed in Table 2-3, and some of the important findings are obtained.

Firstly, the temperatures of the GFRP panel surfaces are literally determined by the environmental conditions and material surface properties. For example, the coefficients of the solar absorptivity and the top surface thermal convection have the decisive effects on the top surface temperatures, and a larger coefficient often leads to significant temperature differences along the panel depth. Secondly, the change of the daily extreme temperature will affect the total heat fields, and it in turn will also influence the bridge temperatures. For example, the increase of the daily maximum temperature increases the top surface temperature and also the temperature differences. Thirdly, the material properties, such as the thermal conductivity, specific heat, and density, have negligible effects on the panel surface temperatures but will determine the bridge conduction abilities. Thus, they will affect the temperature at the middle section and the
temperature differences through the panel depth, even though the effects induced from the material properties are not as considerably important as that from the environmental conditions. Finally, as for all the other parameters, their influences on GFRP panel temperature distributions are insignificant and can be ignored.

### Table 2-3 Temperature Distribution at 12:00, June 23

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Top</th>
<th>Middle</th>
<th>Bottom</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Turbidity</td>
<td>-1.94%</td>
<td>2.03%</td>
<td>-1.14%</td>
<td>1.22%</td>
</tr>
<tr>
<td>Solar absorption</td>
<td>10.53%</td>
<td>-10.65%</td>
<td>5.01%</td>
<td>-5.05%</td>
</tr>
<tr>
<td>Maximum temperature</td>
<td>12.25%</td>
<td>-12.26%</td>
<td>8.89%</td>
<td>-8.80%</td>
</tr>
<tr>
<td>Minimum temperature</td>
<td>1.30%</td>
<td>-1.30%</td>
<td>7.59%</td>
<td>-7.56%</td>
</tr>
<tr>
<td>Conductivity</td>
<td>-0.06%</td>
<td>0.05%</td>
<td>5.15%</td>
<td>-6.24%</td>
</tr>
<tr>
<td>Specific heat</td>
<td>-0.31%</td>
<td>0.28%</td>
<td>-5.47%</td>
<td>7.84%</td>
</tr>
<tr>
<td>Density</td>
<td>-0.30%</td>
<td>0.27%</td>
<td>-5.62%</td>
<td>7.29%</td>
</tr>
<tr>
<td>Top surface convection</td>
<td>-7.80%</td>
<td>10.22%</td>
<td>-3.52%</td>
<td>4.58%</td>
</tr>
<tr>
<td>Bot. surface convection</td>
<td>0.00%</td>
<td>0.00%</td>
<td>0.11%</td>
<td>-0.11%</td>
</tr>
<tr>
<td>Top surface emissivity</td>
<td>-1.30%</td>
<td>1.39%</td>
<td>-0.54%</td>
<td>0.57%</td>
</tr>
<tr>
<td>Bot. surface emissivity</td>
<td>0.00%</td>
<td>0.00%</td>
<td>0.16%</td>
<td>-0.16%</td>
</tr>
</tbody>
</table>

### 2.6 Conclusion

The temperature field of the GFRP honeycomb hollow section sandwich panel is investigated in this paper. Based on the field monitoring program conducted by Kansas DOT, the temperature distribution pattern of the top and bottom surface is observed and discussed. The proposed thermal field finite element model for the GFRP panel, together with the predicted thermal properties and the environmental conditions, is verified by comparing the predicted temperature distributions with that of the field measurements. The effects of the special hollow section configurations and the thermal and environmental parameters on the GFRP panel temperature distributions are further investigated through a parametric analysis. The temperature field results of the present study will provide necessary information to study the thermal stress field of the GFRP panels, and that will be reported separately. Based on the present study, it can be concluded that:

1. Significant temperature differences will be induced on the GFRP panel and they are attributed to the hollow section configurations and the lower thermal conductivities of the GFRP
materials. The top surface’s temperature is highly related to the hourly-varying solar radiation and the bottom surface’s temperature tends to be approaching to that of the ambient due to the air convection.

2. The available temperature design code in AASHTO LRFD (2007) for traditional concrete slabs is no longer valid for GFRP panels since the measured temperature has already exceeded the range that is stipulated in the code. Since the monitored results may not completely represent the worst conditions, it can be estimated that the critical thermal gradients for GFRP panels will be much larger, and the negative temperature gradients may be more important.

3. The thermal gradients for the 12.7 cm (5 in) GFRP panel utilized in the project, though not evident, tend to be nonlinearly distributed; and with the increase of the slab depths, the nonlinear temperature gradients pattern will become more apparent. It should be noted that the monitored temperature distribution only refers to the panel itself, and the temperature distributions of the structures will be different if assembling FRP panels on concrete or steel girders.

4. The heat transfer mechanisms of the inner air convection and mutual radiation within the hollow section will produce less effect on the temperatures of the panel surfaces but significant influences on the thermal gradients through the depths of the panel. In addition, a neglect of the heat transfer mechanisms in hollow section effects will induce larger thermal gradients.

5. The environmental parameters, i.e., daily maximum temperatures, solar radiation (expressed by the material surface absorptivity coefficient), and the wind speeds (expressed by the material surface convection coefficient), are the primary factors determining the temperature distributions of FRP panels; while the material thermal properties only influence the thermal gradients to a small extent.

2.7 References

ANSYS. (2012) ANSYS theory reference, 11.0, ANSYS, Inc.


Meggers, D. A. (2006). Personal communication, Kansas Department of Transportation


CHAPTER 3. TEMPERATURE AND STRESS DISTRIBUTIONS OF GFRP SANDWICH PANEL BRIDGES VERSUS CONCRETE AND STEEL BRIDGES

3.1 Introduction

Temperature variations are significantly important on the performance of bridges and they are often divided into the uniform and gradient components. The former one, referring to the effective average temperature along the cross section of a bridge, will induce the expansion and contraction movements; while the latter one, with varying temperatures from a bridge’s top surfaces to its bottom, will cause the bending deformations. Any restraints to these thermal deformations will induce thermal forces. For example, preventing the uniform temperature movements due to the fixed supports will generate the axial forces and compressive stresses; and the deformations, caused by either the nonlinear temperature gradients through the cross sections or by the different thermal expansion coefficients at the interfacial locations between materials, will develop the internal self-equilibrating stresses. Thus, an accurate estimation of the bridges’ temperature is often fundamental in the aspects of (1) the selection of joints and bearing systems (Arockiasamy et al. 2008; Roeder 2003; Ni et al. 2007), (2) the design of prestressed concrete and steel members (Barr et al. 2005; Shoukry et al. 2009), and (3) the analysis of large segmental and box girder structures (Roberts-Wollman et al. 2002; Tong et al. 2001; Mahama et al. 2009).

Over the recent several decades, with more and more deteriorations of concrete and steel bridges, fiber-reinforced polymer (FRP) composite panels, with the advantages of light weight, high strength, quick installation time, good corrosion resistance, and long-term durability, have been taken as one of the most prosperous alternatives being applied in the fields of the structures’ retrofits and replacements. Generally, the current FRP bridges available in the market are made of glass fibers and polyester or vinyl ester resins, pre-fabricated in the factory, and then delivered and assembled at bridge sites. They are often categorized according to three major manufacturing techniques, including the pultrusion, vacuum-assisted-resin-transfer-molding (VARTM), and open mold hand lay-up (Morcous et al. 2010; Fu et al. 2007). In contrast with the traditional concrete and steel materials, the FRPs have distinctive thermal conduction and expansion abilities; thus, the induced temperature distributions and thermal forces are different from that of the concrete and steel ones. However, there are no such national temperature design guideline for FRP bridges as those specified for the traditional concrete and steel bridges in AASHTO LRFD
(2007), even though some tentative studies were conducted in this aspect (Oghumu 2005; Laosiriphong et al. 2006; Liu et al. 2008; Reising et al. 2004).

In this sense, a review of those proposals during the development of temperature codes for traditional bridges may provide a useful reference in the investigation of the temperature behaviors for FRP bridges. For example, Potgieter and Gamble (1983) proposed an analytical 2D finite difference model for predicting the temperature distribution of a concrete structure. The results of this study laid the foundation for the first publication of the concrete bridge thermal gradient guidelines by the National Cooperative Highway Research Program (NCHRP) (Imbsen et al. 1985). After years of improvements, the temperature design criteria for the concrete and steel bridges are proposed in the AASHTO LRFD (2007) where the maximum and minimum uniform design temperatures are specified by the contours on the map of United States, and the positive and negative thermal gradients are stipulated according to the bridge locations, material types, and superstructure depths.

In this paper, a numerical study is conducted with the purpose to improve and extend the adaptability of the current AASHTO LRFD (2007) temperature design criteria on the FRP bridges. In the study, the adopted FRP panels are in a honeycomb hollow section and sandwich configuration, which is manufactured using the open mold hand lay-up method by the Kansas Structural Composite, Inc. (KSCI). The adoption of this panel is because that many field monitoring studies, experimental tests, and analytical investigations were conducted on them. Thus, the predicted mechanical and physical material properties in the subsequent sections can be justified by comparing the results with the corresponding studies in the literature. It should be noted that, the proposed temperature design guidelines in this study are specific for, but not limited to, the GFRP panel bridges. The results in this research can also provide a meaningful overview for the temperature distribution patterns of other general FRP bridges, and the methods adopted here can provide references for further studies when new FRP materials and configurations are available in the market. Therefore, in this paper, the finite element models are first developed to predict the transient-state thermal fields for the bridges with various materials and configurations, i.e., with the different combining configurations of concrete slabs, GFRP panels, concrete beams, and steel girders. Then, the temperature distribution patterns of the GFRP panel bridges are proposed by comparing them with that of the concrete and steel bridges.
available in the design codes. Finally, the thermal strains and stresses induced from the temperature gradients are examined.

3.2 Temperature Distribution Modeling Method

Procedures for the predictions of temperature distributions have been investigated and well documented for traditional concrete and steel bridges (Elbadry and Ghali 1983; Moorty and Roeder 1992; Mirambell and Aguado 1990; Dilger et al. 1983). Nowadays, the issues encountered during the study of bridges’ temperatures, e.g., intensive computational demand, complicated structure geometrical configurations, and concurrent thermal mechanisms, can be more effectively and conveniently resolved using the numerical simulation method by the commercial finite element programs. In this section, a transient-state thermal field model is firstly developed using the ANSYS 11.0 commercial software. Specifically, the 24-hour temperature field of a one-meter deep concrete slab, an example discussed by Elbadry and Ghali (1983), is simulated with the purpose of examining the available modeling principles and verifying the newly developed modeling methodologies. Some of the important parameters are listed in Table 3-1. For example, similar to that reported by Elbadry and Ghali (1983), the hourly-varying air temperatures are defined by interpolating a sinusoidal curve with the given daily maximum and minimum temperatures, i.e., 30°C (86°F) at 15:00 and 10°C (50°F) at 3:00, respectively; and the solar radiation influx is predicated using an algorithm considering the solar-earth relative positions, solar incident angles, atmosphere clearness conditions, etc. Different from that reported by Elbadry and Ghali (1983), the air convection and solar radiation heat transfer mechanisms, having been jointly accounted by a general heat transfer coefficient, are now separately and conveniently modeled using the surface elements and the corresponding radiation and convection options coded in ANSYS 11.0. In addition, the initial uniform temperature of the bridge is still assumed to be the minimum air temperature at 3:00. In the report, the errors from this assumption is offset by running the program for a 72-hour analysis, or 3 cycling iterations with 24-hour each, under the same environmental conditions; however, this method is changed in the current model where the iteration cycles are increased to 8. The modeling methods and the results are discussed below.
**Table 3-1** FEM Parameters from Elbadry and Ghali (1983)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date</td>
<td>June 21</td>
</tr>
<tr>
<td>Time</td>
<td>01:00 to 24:00</td>
</tr>
<tr>
<td>Location</td>
<td>Calgary, Canada, 51° 03’ N /114° 04’ W</td>
</tr>
<tr>
<td>Elevation</td>
<td>1050 m</td>
</tr>
<tr>
<td>Relative air pressure</td>
<td>0.885</td>
</tr>
<tr>
<td>Turbidity</td>
<td>3.5</td>
</tr>
<tr>
<td>Top/Bot. surface convection</td>
<td>8.5/6 W/m² · K</td>
</tr>
<tr>
<td>Solar emissivity</td>
<td>0.88</td>
</tr>
<tr>
<td>Solar absorptivity</td>
<td>0.5</td>
</tr>
<tr>
<td>Solar constant</td>
<td>1308 W/m²</td>
</tr>
<tr>
<td>Wind speed</td>
<td>1 m/s</td>
</tr>
</tbody>
</table>

Figure 3-1 shows the maximum positive gradients appeared at 15:00 on June 21, after running the program under the same environmental conditions through 1st, 3rd, 5th, 7th and 8th iteration cycles. Also, the results reported by Elbadry and Ghali (1983) are scanned and digitized, shown with the dashed line in Figure 3-1. The predicted results with ANSYS 11.0 match well with that reported by Elbadry and Ghali (1983) when both are running through three iterations. In this sense, the proposed transient-state thermal field finite element modeling method, at least, can be justified by the analytical model introduced by Elbadry and Ghali (1983). However, it can be observed that, with the increase of the iteration cycles, the temperatures at the panel’s top and bottom surfaces do not change as evidently as that through the depth, where the temperature at the middle section of the structure keeps changing until running through five iterations. This phenomenon can be more clearly observed in Figure 3-2, where the temperatures of all 8 cycles’ iterations at the same time of 15:00 on June 21 are plotted for the locations at the slab’s top surface, bottom surface, and middle section. As shown in Figure 3-2, the temperature variations at the top and bottom surfaces are almost constant after 3 cycles’ iterations while they keep increasing at the middle section until 6 ones.

A combined analysis of Figure 3-1 and Figure 3-2 indicates a different temperature converging rate between the internal and external of the concrete slab. This scenario is attributed to the transient thermal field state and the assumed initial uniform temperatures of the bridge. Theoretically, a bridge’s surface temperatures are primarily determined by the environmental conditions caused by the air convection and solar radiation, while the internal ones are affected
by the material conductivities. All three thermal transfer mechanisms are a function of time defined by the material thermal parameters (e.g., conduction and convection coefficients) and environmental conditions (e.g., air temperature, solar influx, and wind speed). In this specific example, then, the thermal energy exchange between the environments and the concrete surfaces is fast with the given environmental parameters; while due to the weaker concrete conduction abilities or the deeper geometrical sizes, more iteration cycles are needed for temperatures to reach the thermal equilibrium state inside the concrete slab. In this sense, an appropriate assumption of the bridge’s initial temperature will be of great importance in the numerical modeling. It may provide a more effective model if choosing the time when the temperature differences within a bridge’s cross section are insignificant, such as during the period between 06:00 and 08:00 recommended by Mirambell and Aguado (1990).

The validated model, together with the environmental conditions, will be further applied in the following parametric study section. However, more iteration cycles will be used for the GFRP panel bridges due to their much weaker conductivity abilities. It should be noted that, for engineering applications, the induced errors from the assumptions of a bridge’s initial temperatures and the iteration cycles may not be significantly influential. Less iteration cycles may induce larger thermal gradients, and that in turn may yield more conservative thermal responses such as the larger thermal strains and stresses.

![Bridge Temperature Gradient](image)

**Fig. 3-1** Thermal Gradients from ANSYS Prediction and Reference by Elbadry and Ghali (1983)
3.3 Parametric Study

In this section, the transient-state thermal field for bridges with different material types and assembling combinations are simulated. The previously introduced environments are revised and utilized again in this analysis as listed in Table 3-2, aiming to compare all the bridge responses under the same environmental conditions. Firstly, the convection coefficients of different materials should have been varying with the surface roughness, material types, wind speeds, etc. However, considering the fact that the effects of the thermal convection behaviors under the current wind speed of 1 m/s (3.28ft/s) are insignificant and show little discrepancies between the different construction materials, then, all the material convection coefficients are assumed to be the same as that of the concrete in this study. Secondly, the solar absorption coefficient of the GFRP panel is assumed to have a larger value than that of the concrete, and this assumption can be justified according to the field monitoring study and the numerical analysis of a GFRP panel bridge reported by Kong and Cai (2012). Thirdly, all the bridge cases are considered to be located at the same place as the GFRP panel bridge, Crawford County, Kansas. Then, the bridge site belongs to the area of zone two, one of the four subdivided solar radiation zones where thermal gradients are correspondingly stipulated in the AASHTO LRFD (2007) temperature design specifications. As for other environmental parameters, i.e., the modeling date and time,
atmospheric turbidity, material’s emissivity, and solar constant, they are also the same as that reported by Elbadry and Ghali (1983).

**Table 3-2 Parameters Used in FEM**

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Concrete</th>
<th>GFRP</th>
<th>Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m²)</td>
<td>2400</td>
<td>1590/1470</td>
<td>7850</td>
</tr>
<tr>
<td>Specific heat (J/kg · K)</td>
<td>880</td>
<td>1415/1486</td>
<td>480</td>
</tr>
<tr>
<td>Conductivity (W/m · K)</td>
<td>1.5</td>
<td>(0.46/0.37)/0.312</td>
<td>54</td>
</tr>
<tr>
<td>Thermal expansion (L/L/°C × 10⁻⁶)</td>
<td>10.8</td>
<td>(12.3/19.7)/20.8</td>
<td>11.7</td>
</tr>
<tr>
<td>Solar absorption</td>
<td>0.5</td>
<td>0.8</td>
<td>N/A</td>
</tr>
<tr>
<td>Emissivity</td>
<td></td>
<td>0.88</td>
<td></td>
</tr>
<tr>
<td>Solar constant (W/m²)</td>
<td></td>
<td>1308</td>
<td></td>
</tr>
<tr>
<td>Atmospheric Turbidity</td>
<td></td>
<td>3.5</td>
<td></td>
</tr>
<tr>
<td>Relative atmospheric pressure</td>
<td></td>
<td>0.968</td>
<td></td>
</tr>
<tr>
<td>Top surface convection (W/m² · K)</td>
<td></td>
<td>8.5</td>
<td></td>
</tr>
<tr>
<td>Bottom surface convection (W/m² · K)</td>
<td></td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>Side surface convection (W/m² · K)</td>
<td></td>
<td>7.5</td>
<td></td>
</tr>
</tbody>
</table>

Note:
a: the two values refer to the GFRP facial laminate and core properties, respectively.
b: the values in the parenthesis are facial laminate’s properties in the x and y directions, respectively; and the value outside of the parenthesis is the core laminate’s property.
c: since only the top surfaces of slabs are to receive solar radiations in this analysis, that the solar absorption coefficients of steel girders are not needed.

Besides the environmental conditions, the other parameters include: (1) three material types, i.e., concrete, steel, and GFRP composite. The isotropic and homogeneous properties of the concrete and steel materials are referred to Elbadry and Ghali (1983) and Fu et al. (1990), while the GFRP composite materials’ properties, determined by the matrix and resin constituents, are predicted using the micro-macro mechanics theory and verified in the next section; (2) four assembling combinations, i.e., concrete slab and concrete beam bridges, concrete slab and steel girder bridges, GFRP slab and concrete beam bridges, and GFRP slab and steel girder bridges. Since the depth of the superstructure is one of the primary factors affecting the vertical temperature gradient distributions in AASHTO LRFD (2007), the varying geometrical sizes of structural elements with 12.7 cm (5in) and 25.4 cm (10in) slabs, type I and type IV concrete beams, and W21×68 and W27×178 steel girders, shown in Figure 3-3 and listed in Table 3-3 and Table 3-4, are considered in the analysis. It should be noted that the adopted environments cannot represent the most critical conditions; however, through the analysis of the thermal
responses of bridges under these parameters, the trends and patterns of the temperature distributions can be clearly obtained.

![Fig. 3-3 Concrete and Steel Girders Geometrical Configurations](image)

**Table 3-3** AASHTO Concrete Beam Section Properties (cm)

<table>
<thead>
<tr>
<th>Type</th>
<th>H_1</th>
<th>H_2</th>
<th>H_3</th>
<th>H_4</th>
<th>H_5</th>
<th>H_6</th>
<th>W_1</th>
<th>W_2</th>
<th>W_3</th>
<th>W_4</th>
<th>W_5</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>71.1</td>
<td>12.7</td>
<td>12.7</td>
<td>27.9</td>
<td>7.6</td>
<td>10.16</td>
<td>40.6</td>
<td>30.5</td>
<td>15.2</td>
<td>12.7</td>
<td>7.6</td>
</tr>
<tr>
<td>IV</td>
<td>137.2</td>
<td>20.3</td>
<td>22.9</td>
<td>58.4</td>
<td>15.2</td>
<td>20.3</td>
<td>66.0</td>
<td>50.8</td>
<td>20.3</td>
<td>22.9</td>
<td>15.2</td>
</tr>
</tbody>
</table>

**Table 3-4** Steel Girder Section Properties (cm)

<table>
<thead>
<tr>
<th>Type</th>
<th>D</th>
<th>B_f</th>
<th>T_f</th>
<th>T_w</th>
</tr>
</thead>
<tbody>
<tr>
<td>W21×68</td>
<td>53.7</td>
<td>21.0</td>
<td>1.7</td>
<td>1.1</td>
</tr>
<tr>
<td>W27×178</td>
<td>70.6</td>
<td>35.8</td>
<td>3.0</td>
<td>1.8</td>
</tr>
</tbody>
</table>

3.3.1 Predication of GFRP Panel Properties

The studied GFRP honeycomb hollowed section sandwich panel is made of E-glass fibers and polyester resins comprising of two facial laminates and one core, as shown in Figure 3-4. The detailed information in terms of the fabrication techniques, geometry descriptions, and constituent layouts can be referred to Cai et al. (2009) and Qiao and Wang (2005). Based on the micro-macro mechanics method (Composite Materials Handbook 2001; McCartney and Kelly 2007), the GFRP material’s thermal and elastic properties are predicted in Table 3-5 and Table 3-6, respectively.
Table 3-5 GFRP Panel Constituent and Laminate Thermal Properties*

<table>
<thead>
<tr>
<th>Material</th>
<th>Density Units</th>
<th>Conductivity W/m K</th>
<th>Specific heat J/kg K</th>
</tr>
</thead>
<tbody>
<tr>
<td>E-glass fiber</td>
<td>2.55 g/cm^3</td>
<td>1.3</td>
<td>840</td>
</tr>
<tr>
<td>Polyester resin</td>
<td>1.14</td>
<td>0.2</td>
<td>1686</td>
</tr>
<tr>
<td>Facial laminate</td>
<td>1.59</td>
<td>0.456/0.365/0.333</td>
<td>1415</td>
</tr>
<tr>
<td>Core mat</td>
<td>1.47</td>
<td>0.308</td>
<td>1486</td>
</tr>
</tbody>
</table>

*Note: The densities of the fiber and resin are given by the manufacture. The coefficients of the conductivity and specific heat are referred to the online material database MATWEB (http://www.matweb.com/) and Bai et al. (2008).

Table 3-6 GFRP Panel Elastic Properties (Oghumu 2005)

<table>
<thead>
<tr>
<th>GFRP panel Unit</th>
<th>E_x Gpa</th>
<th>E_y Gpa</th>
<th>G_xy Gpa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Face laminate</td>
<td>20.15</td>
<td>12.87</td>
<td>3.76</td>
</tr>
<tr>
<td>Core mat</td>
<td>12.65</td>
<td>12.65</td>
<td>4.54</td>
</tr>
</tbody>
</table>

The GFRP’s thermal properties are verified with the field monitoring program conducted by Kansas Department of Transportation. The monitored bridge, located at Crawford County, KS, originally had an asphalt-on-steel deck and was then replaced by the GFRP honeycomb hollow section sandwich panels. The temperature of the panel’s top and bottom surfaces and the ambient were recorded every two hours from December 2002 to July 2004 (Meggers 2006), and the data
during the coldest week from February 7th to February 13th in 2004 is selected for verifications. A transient-state thermal model is developed for the GFRP panel with the corresponding convection and radiation boundary conditions, listed in Table 3-7, according to the local environments at the bridge site. More detailed information of the temperature field modeling can be referred to Kong and Cai (2012). Figure 3-5 and Figure 3-6 show the comparisons of the predicted and measured temperatures at the bottom and top surfaces of the GFRP panel, respectively. The well matched results verify the accuracy of the predicated material thermal properties.

### Table 3-7 FEM Parameters for Crawford County Bridge

<table>
<thead>
<tr>
<th>Date</th>
<th>Feb. 7th to Feb. 13th, 2004</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time</td>
<td>02:00 to 24:00</td>
</tr>
<tr>
<td>Location</td>
<td>Crawford County, KS, 38° N/95°W</td>
</tr>
<tr>
<td>Air Pressure</td>
<td>0.968</td>
</tr>
<tr>
<td>Atmospheric turbidity</td>
<td>1.8</td>
</tr>
<tr>
<td>Top/Bot. surface convection</td>
<td>15 W/m²·K</td>
</tr>
<tr>
<td>Top/Bot. solar emissivity</td>
<td>0.5/0.3</td>
</tr>
<tr>
<td>Solar absorption</td>
<td>0.8</td>
</tr>
</tbody>
</table>

**Fig. 3-5** Bottom Surface Temperature Comparisons between the Measured and Predicted
Fig. 3-6 Top Surface Temperature Comparisons between the Measured and Predicted

The GFRP material’s elastic properties are verified by referring to the static experimental test conducted at West Virginia University (Robinson 2001) where a 4.448 kN (1000lb) force was applied at the mid-span of four simply supported and longitudinally oriented GFRP beams, with varying lengths of 4.57m (180in), 3.51m (138in), 2.43m (96in), and 1.67m (66in), respectively. The measurements, as marked in Figure 3-7, were recorded for the beam deflections at the 1/3, 1/2, and 2/3 span length locations, the strains of the beam’s top surface with a 0.15 m (6in) distance away from the loading location, and the strains of the beam’s bottom surface at the mid-span. For modeling this complicated GFRP sinusoidal configuration shown in Fig. 4, the simplified one-layer or three-layer method, as typically used to predict the global behaviors, such as deflections, is no longer suitable since the local thermal gradients and stresses through the depths, rather than the global responses, are now needed. Therefore, a full 3D model using solid elements for the actual geometry is developed as shown in Figure 3-8. Table 3-8 and Table 3-9 illustrate the comparisons between the predicted and tested results in terms of the deflections and strains. A good agreement is observed between these results with the differences generally being less than 10%. Thus the predicted GFRP elastic properties are validated.
**Fig. 3-7** Load Tests Conducted at West Virginia University (Robinson 2001)

**Fig. 3-8** Sketch of the GFRP Beam Finite Element Model

**Table 3-8** Deflection Comparisons of the FEM and Experiment Results

<table>
<thead>
<tr>
<th>Cases</th>
<th>L/3</th>
<th>L/2</th>
<th>2L/3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam(m)</td>
<td>FEM</td>
<td>EXP</td>
<td>Diff%</td>
</tr>
<tr>
<td>4.57</td>
<td>0.52</td>
<td>0.48</td>
<td>8.40</td>
</tr>
<tr>
<td>3.51</td>
<td>0.23</td>
<td>0.22</td>
<td>7.85</td>
</tr>
<tr>
<td>2.43</td>
<td>0.08</td>
<td>0.07</td>
<td>9.49</td>
</tr>
<tr>
<td>1.67</td>
<td>0.03</td>
<td>0.03</td>
<td>-1.19</td>
</tr>
</tbody>
</table>
### Table 3-9 Strain Comparisons of the FEM and Experiment Results

<table>
<thead>
<tr>
<th>Cases</th>
<th>Top_1</th>
<th>Top_2</th>
<th>Top_3</th>
<th>Mid-Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam (m)</td>
<td>FEM</td>
<td>EXP</td>
<td>Diff %</td>
<td>FEM</td>
</tr>
<tr>
<td>4.57</td>
<td>-200</td>
<td>-183</td>
<td>9</td>
<td>-202</td>
</tr>
<tr>
<td>3.51</td>
<td>-150</td>
<td>-136</td>
<td>11</td>
<td>-151</td>
</tr>
<tr>
<td>2.43</td>
<td>-100</td>
<td>-90</td>
<td>12</td>
<td>-100</td>
</tr>
<tr>
<td>1.67</td>
<td>-64</td>
<td>-57</td>
<td>12</td>
<td>-64</td>
</tr>
</tbody>
</table>

### 3.3.2 Discussion of Temperature Gradients

The temperature distribution results for all the parametric cases introduced above are investigated in this section. A consistent legend convention is firstly defined and used throughout the following discussions where the four characters successively denote the slab material types, i.e., C (concrete), G (GFRP); the slab depths, i.e., 5 (12.7 cm or 5in), 10 (25.4 cm or 10in); the beam material types, i.e., C (concrete), S (steel); and the beam types, i.e., 1 (type I concrete or W21×68 steel), 4 (type IV concrete or W27×178 steel). For example, the C5S1 case means the bridge is a 12.7 cm (5in) high concrete slab and a W21×68 type steel beam.

The representative temperature gradients at 15:00 on June 21 for bridge cases with the highest and lowest superstructure depths are plotted where the temperature values are normalized by subtracting the minimum value from the absolute ones along the panel depth. At the same time, since all the bridge cases are assumed to be located in Kansas, then the temperature gradients, specified for concrete and steel bridges in this region from the AASHTO LRFD (2007), are also plotted in the corresponding concrete and steel girder figures for the convenience of comparisons and discussions.

Figure 3-9 shows the thermal gradients for the cases with concrete slabs and concrete beams where the zero depth is designated as the slab-beam interface location. It can be observed that the temperatures near the top and bottom surfaces of the superstructure have apparent gradients, and it is attributed to the influences of the adjacent environments, such as solar radiation and air convection. However, the high energy at the surfaces can only transfer approximately 25.4 cm (10in) through the concrete depths due to the lower concrete thermal conductivities, no matter how deep or shallow the total superstructure size is. Thus, most of the concrete web areas show zero temperature variations. In addition, the change of the
superstructure depths have negligible effects on the thermal distribution patterns and the multi-linear thermal gradient distributions specified in the AASHTO LRFD (2007) are justified.

![Graph showing thermal gradients](image)

**Fig. 3-9** Thermal Gradients of the C5C1 and C10C4 Cases

Figure 3-10 shows the temperature gradients for the GFRP slab and concrete beam cases. Generally, they have very similar patterns to that of the concrete slab and concrete beam cases, as shown in Figure 3-9. For example, the temperatures near the bottom of the concrete beam are affected by the nearby environments, and the temperatures are almost constantly distributed with a zero gradient through the concrete beams. However, the temperature distribution patterns at the slab areas of the superstructure between these two cases are different. Due to the much weaker thermal conduction abilities of the GFRP panels, originated from the lower material thermal conductivities and hollowed section configurations, the high heat energy at the panel’s top surface could hardly travel through the GFRP slab. The result of this behavior leads to almost no temperature transitional segments between the GFRP panel and the concrete beam near the zero depth location, and this scenario is different from the concrete slab and concrete beam cases as shown in Figure 3-9. Moreover, the predicted panel’s top surface temperatures in this project, though not the worst cases, are almost exceeding the maximum values as specified in the AASHTO LRFD (2007) for concrete slab and concrete beam bridges.
Figure 3-11 shows the temperature gradients for bridges with concrete slab and steel beam cases. It can be observed that the nonlinear temperature differences only develop in the concrete slabs while almost uniformly distributed through the rest steel girder depths. Contrary to the full concrete superstructure cases shown in Figure 3-9, the slab depths now have obvious effects on the temperature transfer patterns, and two different scenarios are clearly observed in Figure 3-11 for the shallower and deeper concrete slabs. Generally, during the daytime around the mid-day, the whole bridge is heated up by external environments, and the temperature of the steel girders can be increased to that of the air very quickly due to their larger thermal conductivity coefficients and thin web geometries. However, it is not the case for the concrete slabs where the abilities of the solar radiations travelling downwards from the top surfaces will depend on the slab depths. For a shallower concrete slab, the high energy is easily transferring through the slab and arriving at the steel girder’s top surface, and it in turn will change and heat up the steel girder. In this case, the temperature transferring behavior, from the top to the bottom, will not induce temperature differences on the steel girders as in the C5S1 case shown in Figure 3-11. For a deeper concrete slab, nevertheless, the higher top surface temperature cannot arrive at
the steel girder’s top surface. Thus, the temperature differences between the higher steel girders and lower concrete slabs will induce a reverse conduction from the bottom to the top. As a result, the uniform temperature distributions are developed, and the minimum temperature appears at the lower part of the concrete slab, around the zero depth location, as in the C10S4 case shown in Figure 3-11. Moreover, the multi-linear temperature distributions specified in the AASHTO LRFD (2007) for steel girder bridges can approximately describe the two thermal transfer behaviors.

![Fig. 3-11 Thermal Gradients of the C5S1 and C10S4 Cases](image)

Figure 3-12 shows the thermal gradients for the GFRP slab and steel beam bridge cases. It can be observed that the temperatures are distributed linearly at the GFRP panels and uniformly through the steel girders. Similar to the previous concrete slab and steel girder cases in Figure 3-11, two different temperature transferring behaviors appear with the change of superstructure’s depths, as shown in Figure 3-12. Similar to the GFRP slab on concrete beam cases shown in Fig. 10, the predicted top surface temperatures and the girder thermal gradients, as the G10S4 and G5S1 cases shown in Figure 3-12, are almost reaching or exceeding the design temperature specified for concrete slab and steel girder bridges in AASHTO LRFD (2007).
Therefore, it can be implied from the above observations that, with the same beam types, the bridge thermal gradients between the GFRP panels and concrete slabs actually have very similar temperature distribution patterns. Through a further comprehensive observation of all the studied cases, shown in Figure 3-13 and Figure 3-14, together with the detailed information in terms of the predicted maximum and minimum temperatures and the maximum temperature differences along the bridge depths, listed in Table 3-10 and Table 3-11, the temperature distribution patterns for the GFRP composite panel bridges can be proposed in a similar format referring to the AASHTO LRFD (2007) as:

1. The thermal gradients of bridges with GFRP slabs and concrete beams can be obtained by revising that of the concrete beam and concrete slab bridges. Through comparisons between these two cases, shown in Figure 3-13, the temperature distributions near the bottom parts of the concrete beam are almost the same, while the temperature gradients near the top slabs tend to be more linearly distributed in the GFRP slabs and different from the nonlinear ones in the concrete slabs due to the much lower GFRP thermal conductivities. In addition, Table 3-10 shows that the temperature at the concrete slab’s top surface is about 17°C (30°F) less than that at the GFRP slab due to the different materials’ solar absorptivity. Therefore, the existing four-segment-line concrete bridge thermal gradient criteria specified in AASHTO LRFD (2007) can be revised into
a three-segment-line one for the GFRP slab and concrete beam bridges, consisting of one linear segment from the GFRP panel’s top surface to the concrete beam’s top surface, one linear segment for the bottom part of the concrete beam the same as that in the AASHTO LRFD (2007) code, and one constant segment with a zero gradient through the rest middle beam web. However, the GFRP panel’s top surface temperature should be further determined based on the material’s actual solar absorption abilities.

2. Similarly, the thermal gradients of bridges with GFRP slabs and steel girders can be adapted from that of the concrete slab and steel girder bridges. As is observed from Figure 3-14, the temperatures’ distribution patterns at the steel beams are consistent for the two cases, and only the thermal gradients at the GFRP panel’s top parts need to be revised. Therefore, a linear segmental line can be proposed from the GFRP panel’s top surface to the steel beam’s top surface. The thermal gradient pattern in steel girders is still similar to that specified in AASHTO LRFD (2007), but a larger value should be used. Also, the results from Table 3-11 again imply that the temperature values at the GFRP panel’s top surface should be increased based on the material’s actual absorption abilities.

Fig. 3-13 Thermal Gradients of Bridges with Concrete Beam Cases
Table 3-10 Temperature Comparisons for Concrete Beams with GFRP and Concrete Slabs

<table>
<thead>
<tr>
<th>Case</th>
<th>Max (F)</th>
<th>Min (F)</th>
<th>Diff. (F)</th>
<th>Case</th>
<th>Max (F)</th>
<th>Min (F)</th>
<th>Diff. (F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C5C1</td>
<td>101.85</td>
<td>69.272</td>
<td>32.578</td>
<td>G5C1</td>
<td>131.78</td>
<td>68.923</td>
<td>62.857</td>
</tr>
<tr>
<td>C5C4</td>
<td>101.7</td>
<td>69.307</td>
<td>32.393</td>
<td>G5C4</td>
<td>131.92</td>
<td>67.724</td>
<td>64.196</td>
</tr>
<tr>
<td>C10C1</td>
<td>101.86</td>
<td>68.859</td>
<td>32.393</td>
<td>G10C1</td>
<td>131.78</td>
<td>68.908</td>
<td>62.872</td>
</tr>
<tr>
<td>C10C4</td>
<td>101.7</td>
<td>68.863</td>
<td>32.837</td>
<td>G10C4</td>
<td>131.92</td>
<td>67.794</td>
<td>64.126</td>
</tr>
</tbody>
</table>

Table 3-11 Temperature Comparisons for Steel Girders with GFRP and Concrete Slabs

<table>
<thead>
<tr>
<th>Case</th>
<th>Max (F)</th>
<th>Min (F)</th>
<th>Diff. (F)</th>
<th>Case</th>
<th>Max (F)</th>
<th>Min (F)</th>
<th>Diff. (F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C5S1</td>
<td>104.02</td>
<td>85.576</td>
<td>18.444</td>
<td>G5S1</td>
<td>131.85</td>
<td>84.756</td>
<td>47.094</td>
</tr>
<tr>
<td>C5S4</td>
<td>101.98</td>
<td>80.902</td>
<td>21.078</td>
<td>G5S4</td>
<td>131.91</td>
<td>74.941</td>
<td>56.969</td>
</tr>
<tr>
<td>C10S1</td>
<td>103.51</td>
<td>84.726</td>
<td>18.784</td>
<td>G10S1</td>
<td>131.83</td>
<td>83.125</td>
<td>48.705</td>
</tr>
<tr>
<td>C10S4</td>
<td>101.84</td>
<td>80.164</td>
<td>21.676</td>
<td>G10S4</td>
<td>131.91</td>
<td>74.648</td>
<td>57.262</td>
</tr>
</tbody>
</table>

3.4 Discussion of Thermal Strain and Stress Results

Having obtained the temperature gradients, the thermal strains and stresses for all bridge cases discussed above can be calculated accordingly. The analytical derivations conform to two assumptions: (1) material properties are independent of temperatures and (2) the Navier–
Bernoulli hypothesis that initial plane sections remain plane after subjected to thermal loadings (Priestly 1978).

Theoretically, for externally determinate girders as shown in Figure 3-15, structures subjected to the nonlinear temperature gradients will expand freely if there are no restraints. The thermal strains will be induced as:

\[ \varepsilon_{\text{thermal}} = a \Delta T_y \]  

(1)

where \( a \) is the material linear thermal expansion coefficient, and \( \Delta T_y \) is the temperature gradients. Based on the Navier–Bernoulli plane section hypothesis, the total strains should be in a linear distribution as:

\[ \varepsilon_{\text{total}} = \varepsilon_0 + y \varphi_0 \]  

(2)

where \( \varepsilon_0 \) is the strain at the centroid of the cross section and \( \varphi_0 \) is the curvature. Therefore, the differences between the free thermal strain and the total strain will be the restraint strain, or the elastic strain, and that in turn will induce thermal stresses as:

\[ \sigma_y = E(\varepsilon_{\text{total}} - \varepsilon_{\text{thermal}}) \]  

(3)

where \( E \) is the material Young’s modulus. Through integrating the stresses over the cross section and taking the moment about the section’s neutral axis, the axial force and moment are yielded. For a simply supported beam, the internal force and moment are zero. Thus, the variables of \( \varepsilon_0 \) and \( \varphi_0 \) can be obtained. For an externally indeterminate structure, the restraints from the supports will induce secondary moments, and the total stresses will be obtained by adding the stresses from the secondary moments. Therefore, if given the thermal gradients, the thermal strains and stresses can be calculated based on the Eq. (1) to Eq. (3). In the following sections, the thermal strains and stresses induced from the temperature gradients at 15:00 on June 21 as predicted earlier will be discussed.
3.4.1 Thermal Strain Results

Figure 3-16 shows the representative strain distributions from the four cases where the total strain, thermal strain, and elastic strain distribution patterns described in the analytical method are verified. Figure 3-16 shows that the C5C1 case has evident nonlinear strains through the whole superstructure depth, while the other three cases, C5S1, G5C1, and G5S1, only have that kind of variations at the top slab parts but with almost constant or linear ones though the beams. In addition, strain discontinuities may develop at the interfaces between two materials with different thermal properties. For example, at the interfaces between the concrete and steel materials as in the case of C5S1, due to the similar material thermal expansion coefficients, the induced strain discontinuity is negligible; while in the cases of G5C1 and G5S1, the different material properties lead to apparently sudden strain changes.
Figure 3-17 shows the strain distributions for the cases with concrete beams. For concrete slab cases, with the change of superstructure depths, the strain distribution patterns remain unchanged. In addition, the induced strains are primarily coming from the nonlinear thermal gradients, and the maximum positive strain values may appear at the maximum temperature location in the concrete beam. For GFRP slab cases, however, the strain patterns are mainly affected by the material property differences, i.e., different thermal expansion coefficients between the GRFP and concrete materials. Thus, the maximum values appear at the two material interfaces. Moreover, under the same environmental conditions, replacing the concrete slab with a same height GFRP panel will generate both larger positive and negative thermal strains as the values shown in Figure 3-17.

![Fig. 3-17 Strain Distributions of Concrete Beam Cases](image)

Figure 3-18 shows the strain distributions for the steel girder cases. Obviously, the strain distributions are not necessarily showing a similar pattern with the varying superstructure depths. Taking the concrete slab cases in Figure 3-18 for an example, opposite strain signs are observed between the deeper concrete slab cases, i.e., C10S1 and C10S4, and the shallower ones, i.e., C5S1 and C5S4. In addition, the maximum positive thermal strains appear in the concrete slabs and should be given enough attentions in case any tensile forces induce concrete cracks. Moreover, similar to the previous concrete beam cases in Figure 3-17, replacing concrete slabs
with GFRP panels for steel girder bridges under the same environmental conditions will also induce evidently larger strains, as shown in Figure 3-18.

![Strain Distributions of Steel Girder Cases](image)

**Fig. 3-18 Strain Distributions of Steel Girder Cases**

### 3.4.2 Thermal Stress Results

In the applications of FRP composite bridges, one of the concerns is on the bridge performances after slab replacements. Thus, this section discusses the induced thermal stresses of concrete and steel girders after replacing the concrete slabs with the same height GFRP panels. In the modeling, the behaviors between the slabs and beams are assumed to be fully composite for original concrete and steel bridges before slabs’ replacements. This assumption is reasonable for traditional bridges but may be inappropriate for GFRP composite bridges. In practice, the connections between the GFRP panel and concrete or steel beams are often implemented by using epoxy adhesives or special clamp devices so that these bridges are often acting in partial-composite or non-composite behaviors. In this study, the thermally most critical condition, full-composite action behavior, is assumed for GFRP panel bridges after the slab’s replacement. The maximum positive self-equilibrating thermal stresses, induced from the temperature gradients through the beam depths at the bridge’s mid-span, are compared in Figure 3-19 and Figure 3-20, respectively. All stresses are normalized by dividing the corresponding material strength, i.e.,
\( f_{\text{concrete}} = 40\,\text{mpa} (5,800\,\text{psi}) \) and \( f_{\text{steel}} = 345\,\text{mpa} (50\,\text{ksi}) \), where the positive sign, shown in these figures, is designated for the ratio between tension stress and material strength.

Figure 3-19 shows the comparisons of the thermal stresses for the concrete beam cases. First, for all the concrete bridges before slab replacements, the induced thermal stresses tend to decrease slightly with the increase of the superstructure depths. For all the GFRP panel bridges after replacements, however, the stresses vary comparatively obviously with the change of the superstructure’s depths, and the larger thermal stresses appear in the cases with the shallower concrete beams, i.e., G5C1 and G10C1. Second, it can be observed from each pair of slab replacement cases that the thermal stresses are generally increased except for the G5C4 case, and the induced maximum stress on the concrete beam increases about 5% in the G10C1 case.

It should be noted that the stress generation mechanisms are actually different for concrete and GFRP slab bridges. For the original concrete ones, the thermal stresses are mainly induced from nonlinear thermal gradients; while for the replaced GFRP ones, the thermal stresses are mainly from the deformation incompatibility at the interfaces due to the difference of material properties. In this sense, if the connection behaviors between the slab and girder are non-composite after panel’s replacements, the induced thermal stresses on the beams may be negligible; while if the connections are partial or full composite, a comparatively large thermal stress may be produced. As the cases discussed in this study, the induced maximum thermal stress on the concrete beams after the GFRP panel’s replacements, while not the worst condition, is already as high as 10% of the material’s compressive strength in the case of G10C1, which may result in cracks when combining with other loading effects.

![Fig. 3-19 Stress Comparisons between Concrete and GFRP Slabs for Concrete Beam Cases](image-url)
Figure 3-20 shows the comparisons of thermal stresses for all steel girder cases. Obviously, before concrete slab replacements, there are almost no, if any, thermal stresses along the steel beams due to the linear or constant temperature distribution patterns as discussed earlier. In addition, the stress in the C10S1 case is observed to show an opposite sign compared to the other three cases. This phenomenon can be attributed to the change of the superstructure’s depths, as discussed in the previous temperature and strain sections shown in Figure 3-14 and Figure 3-18, respectively.

After the GFRP panel replacements, however, the induced stresses vary. Similar to the previous concrete girder cases in Figure 3-19, the largest stresses again develop in the cases with deeper GFRP panels, i.e., G10S1, or shallower steel beams, i.e., G5S1. However, contrary to that shown in Figure 3-19, where the apparent stress increase may only happen in the G10C1 replacing C10C1 case, Figure 3-20 shows that replacing concrete slabs with GFRP panels on steel girder bridges will generate evident stress increases for all the four cases, though the percentage is less than 6% of the steel strength.

![Fig. 3-20 Stress Comparisons between Concrete and GFRP Slabs for Steel Girder Cases](image)

3.5 Conclusion

In this paper, the thermal behaviors of GFRP composite, concrete, and steel bridges are investigated. A transient-state thermal field finite element model is firstly developed and discussed where the GFRP panel’s mechanical and physical properties are predicted with the micro-macro mechanics theory and verified with a field monitoring program and an experimental test. Then, a parametric study is conducted to analyze the thermal responses of bridges with
different material and geometrical configurations under the same environmental conditions. Results from the research can be concluded as:

1. The proposed finite element modeling method using ANSYS 11.0 is effective on the investigation of the bridges’ thermal performance. The initial uniform temperature assumptions of bridges in the numerical modeling are important and could largely affect the predicted temperature gradients. When modeling GFRP panel bridges, more iteration cycles are required due to their lower conduction coefficients and hollowed section configurations.

2. For bridges with a GFRP slab and concrete beams, the temperature gradients can be obtained by revising the current AASHTO LRFD (2007) design codes for concrete bridges, where three segmental linear distributions can be proposed, including one linear segment from the GFRP panel’s top surface to the top of the concrete beam, one linear segment the same as that specified in AASHTO LRFD (2007) code near the concrete beam bottom, and one constant segment with a zero gradient through the rest beam depth. The temperature value at the GFRP panel’s top surface needs to be determined according to the material solar absorptivity.

3. Similarly, for bridges with a GFRP slab and steel beams, the temperature gradients can be referred to the existing design criteria for steel bridges in AASHTO LRFD (2007), where two segmental linear distributions are defined with one linear segment from the GFRP panel’s top surface to the steel girder’s top surface and a constant temperature distribution, with a larger value compared to that specified in the AASHTO LRFD (2007) code, through the steel girders.

4. Strain distribution patterns are different for all cases, and they are generally determined by either of the two reasons: (1) nonlinear temperature gradient distributions and (2) differences of material’s thermal expansion coefficients. For GFRP panel bridges, the thermal strains caused by the first factor is negligible since the weaker thermal conductivity of the GFRP panels prevents the high temperatures from transferring through depths, and that, in turn, leads to only linear or constant temperature distributions. Then, the thermal stresses of GFRP bridges are primarily coming from the second reasons. Since the thermal expansion coefficients of GFRP bridges are not only different from that of the concrete and steel bridges, but also different within the GFRP panel itself, e.g., the facial and core laminates of the GFRP panel, large thermal stresses are observed to be generated at the interfaces between these materials.

5. For concrete slabs, with the change of superstructures’ depths, the thermal stresses on the concrete beams or steel girders have insignificant variations for all cases. After replacing the
concrete slab with a same height GFRP slab, however, the induced positive thermal stresses are
increased for both cases. If the connection behaviors between slabs and beams are partial or full
composite, the induced thermal stresses cannot be neglected. As the cases discussed in this study,
even though the environmental condition is not the worst, the induced thermal tensile stresses on
the concrete beam and steel girder after replacements with GFRP panels are increasing almost 5%
of the concrete compression strength and 6% of the steel strength, respectively.

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CHAPTER 4. THERMAL BEHAVIORS OF CONCRETE AND STEEL BRIDGES AFTER SLAB REPLACEMENTS WITH GFRP SANDWICH PANELS

4.1 Introduction

In bridge engineering, composite superstructure with the fiber-reinforced polymer (FRP) materials has been applied in lieu of the traditional concrete and steel ones during the last several decades. One of the primary reasons of their applications is originated from the huge maintenance expenditure on the rapidly deteriorated structures, especially those caused by the severe environmental conditions, overweight loadings, aging of materials, etc. For example, as indicated by the Federal Highway Administration (FHWA), approximately 31.4% (180,000 out of 580,000) bridges in the United States are classified as structurally deficient or functionally obsolete and the maintenance cost is estimated more than $20 billion. In this sense, the FRP panels, with the benefits of high strength, light weight, long-term durability, and good corrosion and fatigue resistances, have been considered as one of the best alternatives for the slabs’ retrofits and replacements (Plunkett 1997; Alampalli et al. 2002).

In the year of 1996, the first highway composite bridge was built in the state of Kansas and more than one hundred of such bridges with different configurations have been designed and constructed thereafter in the USA. However, due to the lack of national design guidelines, together with the concerns on the initial material costs and long-term performances, the further applications of FRP bridges are restricted. In this sense, a plenty of laboratory experiments, field tests, long-term monitoring programs, and numerical modeling studies have been conducted to investigate their behaviors under their distinctive physical and mechanical properties. For example, (a) a state-of-the-art of the FRPs’ development, present and future utilizations, and in-service material properties have been well reviewed (Hollaway 2010; Bakis et al. 2002); (b) the modeling and design methods, particularly that aiming to simplify the complex geometry configurations of the FRP panels, have been proposed including the one-layer model, three-layer model, simplified I-beam model, and actual configuration model (Cai et al. 2009; Davalos et al. 2001; Morcous et al. 2010); (c) the static tests on the structure overall responses have been implemented to analyze the strength and rigidity characteristics, live load distribution factors, and composite action behaviors (Camata and Shing 2005; Alagusundaramoorthy et al. 2006; Zhang and Cai 2007; Turner et al. 2004); and (d) the dynamic analyses have been performed to
study their unique behaviors in terms of the dynamic allowance factors, natural frequencies, damping ratios, and deck accelerations (Zhang et al. 2006; Aluri et al. 2005). However, the study on the thermal behavior of FRP bridges is not adequate, where very few studies have been focused on their responses under the natural environments (Laosiriphong et al. 2006; Liu et al. 2008; Reising et al. 2004) and several others on that under the extreme fire or freeze-thaw conditions (Alnahhal et al. 2006; Wu et al. 2006). Nevertheless, the thermal behaviors of FRP bridges are of significant importance. They may largely influence the behaviors of a bridge in terms of the overall and local deflections, the connecting behaviors at the interfaces between the panel-panel, panel-beam, and also the joints and bearing systems. More importantly, the induced thermal deformations and stresses, sometimes being comparable to that of the live loads, may also cause the detrimental damages on other structural elements.

Therefore, this paper demonstrates a numerical study on a glass FRP (GFRP) bridge based on a field study program conducted by Kansas State Department of Transportation (DOT) in which a live load test was performed and the long-term bridge and environmental temperatures were measured. First, a numerical finite element model, using a sub-structuring method, is proposed with the help of the commercial software ANSYS 11.0 and verified by comparing the predicted live load distribution factors with that of the live load tests. Second, the thermal behaviors of the bridge are discussed using the measured temperatures from January 24 to July 13 in 2004. Finally, a parametric study is employed to compare the behaviors of two general slab replacement cases, i.e., replacing concrete slabs with GFRP panels for both a concrete beam and a steel girder bridges, respectively. The induced deformations and stresses under the uniform temperature variations, nonlinear thermal gradients, dead loads, and HS-20 live loads are examined.

4.2 Project Study

The studied bridge, shown in Figure 4-1, is located at Crawford County, Kansas. It originally had the asphalt-on-steel decks supported by fourteen AISC W21×68 I-beam stringers, and then the decks were replaced by the GFRP honeycomb hollow section sandwich panels. The total bridge is 13.7m (45ft) long and 9.8m (32ft) wide with five 9.8m × 2.7m (32ft × 9ft) GFRP panels lying across the longitudinal steel girders. The panels were made of glass fibers and polyester resins, manufactured through a hand lay-up technique by Kansas Structural Composites,
Inc. (KSCI). Specifically, the detailed configuration of the panel, shown in Figure 4-2, constitutes with a series of repeatedly and periodically occurring units, called representative volume element (RVE); and each one RVE includes two stiff facial laminates and one light weight core.

![Fig. 4-1 GFRP Honeycomb Hollow Section Sandwich Panel Bridge](image1)

![Fig. 4-2 Sketch of the GFRP Panel Configuration](image2)

### 4.2.1 Live Load Test

The live load test was conducted by Kansas DOT with the purpose to compare the lateral load distribution performance at the mid-span between the original metal and the replaced GFRP panels. In the present study, the testing results are used for the verifications of the proposed modeling method. The testing schemes, referred to Zhang and Cai (2007) and Schreiner and Barker (2005), are shown in Figure 4-3. Totally, ten passes of HS-20 trucks, shown in Figure 4-4,
were performed on the bridge slab with the first and tenth loading positions plotted in Figure 4-3 and the other eight ones were laterally and equally distributed across the bridge width. The lateral distribution factor is calculated as the ratio between the stresses induced from the HS-20 truck on one particular girder and that on the sum of all girders referring to AASHTO LRFD (2007), considering the number of loaded trucks, expressed as:

\[
DF_i = n \frac{\sigma_{\text{girder}_i}}{\sum_{j=1}^{14} \sigma_{\text{girder}_j}}
\]

where \(DF_i\) = distribution factor for girder \(i\); \(n\) = number of trucks used in the test for producing a maximum effect along a bridge section; and \(\sigma_{\text{girder}_i}\) = stress of girder \(i\).

Fig. 4-3 HS-20 Truck Loading Positions

Fig. 4-4 HS-20 Truck Information
4.2.2 Sub-structuring Modeling Method

In modeling of such a complex sinusoidal honeycomb and hollowed section sandwich panel, the traditional method based on the actual configuration is nearly impossible since it will generate a huge amount of elements and the computation will be overwhelmingly time consuming. For example, Zhang and Cai (2007) indicated that, if modeling a small size of such panel, i.e., $4.6 \text{m} \times 2.3 \text{m} \times 12.7 \text{cm} (15\text{ft} \times 7.5\text{ft} \times 5\text{in})$, and a minimum of four elements being used for one sine wave plate, it will require 133200 shell elements, let alone modeling the whole bridge with both the panel and girder structures. Likewise, other simplified modeling methods, based on the stiffness equivalent theory, are also inapplicable in this study. These methods may be appropriate in the study of the global behaviors of a bridge, i.e., global displacements, but not for the thermal responses. The local information is now required through the depth of the superstructure, such as the nonlinear temperature loadings and the corresponding thermal strains and stresses. Under this circumstance, an alternative sub-structuring modeling method is adopted. This method enables to condense a group of finite elements into one superelement and the superelement will be used in the subsequent analysis. The details of this method can be referred to the manual of ANSYS.11.0 and the important points are briefly illustrated below for the convenience of readers.

Generally, the basic stiffness equation for a static problem is expressed as:

$$[K][U] = [F]$$  \hspace{1cm} (2)

where $[K]$ is the stiffness matrix, $[U]$ is the displacement vector, and $[F]$ is the force vector. This equation can further be partitioned into two groups, expressed as Eq. (3), including one group with the master degrees of freedoms (DOFs), denoted by a subscript $m$, and the other with the slave DOFs, denoted by a subscript $s$, respectively. The former one is defined at those necessary nodes and DOFs, e.g., nodes where results will be acquired, loading and boundary locations will be applied, interfacial connections between the superelements or between the superelements and normal elements, etc.; while the later one is defined for all the other unnecessary nodes and DOFs.

$$\begin{bmatrix} [K_{mm}] & [K_{ms}] \\ [K_{sm}] & [K_{ss}] \end{bmatrix} \begin{bmatrix} [U_m] \\ [U_s] \end{bmatrix} = \begin{bmatrix} [F_m] \\ [F_s] \end{bmatrix}$$  \hspace{1cm} (3)
Eq. (3) can be further manipulated and expanded as:

\[
\begin{bmatrix}
  [K_{mm}] - [K_{ms}][K_{ss}]^{-1}[K_{sm}]
\end{bmatrix}
\{U_m\} = \{F_m\} - [K_{ms}][K_{ss}]^{-1}F_s \tag{4}
\]

or,

\[
[K^*] \{U_m\} = \{F^*\} \tag{5}
\]

\[
[K^*] = [K_{mm}] - [K_{ms}][K_{ss}]^{-1}[K_{sm}] \tag{6}
\]

\[
\{F^*\} = \{F_m\} - [K_{ms}][K_{ss}]^{-1}F_s \tag{7}
\]

where \([K^*]\) and \(\{F^*\}\) are the generalized stiffness matrix and force vector of the superelements, respectively. Additionally, a more powerful sub-structuring modeling feature is the ability to use the nested superelements, i.e., when generating a superelement, one of the elements in this generation step is the previously generated superelement. Therefore, through this procedure, by defining the superelements, the whole complicated stiffness matrix will be condensed into a small one only with the master DOFs. Then, the generated superelements with condensed DOFs will be used in the analysis; and the calculation time will be significantly saved. Moreover, this method is especially suited for very large structures with repeated geometrical patterns, e.g., the GFRP panels with repeatedly appearing RVE units in this study, where it can generate one or more layers of superelements to represent the repeated patterns and simply make copies of them at different locations. Therefore, the sub-structuring modeling method is implemented in three steps, including, (1) generating the stiffness matrix for superelements; (2) using the generated stiffness matrix in the calculations to obtain the results at the master DOFs; and (3) expanding the condensed matrix to obtain the results at the other slave DOFs if needed.

Figure 4-5 shows the detailed modeling procedures for this GFRP panel bridges subjected to the distributed live loads. Taking the shaded 8m × 2.8m (26ft × 9ft) area as shown in Figure 4-5(a) for example, the full finite elements are firstly divided into several areas with the same repeating patterns such in Figure 4-5(b). Then, a typical pattern area, with the size of 2m × 1.4m (13ft × 4.5ft), is selected and the DOFs of that area are condensed to generate the first layer superelement. This superelement is further copied to all the other locations as the area with dashed lines shown in Figure 4-5(b). Following this procedure, from Figures 4-5(c) to 4-5(e), the
superelements generated in the previous steps will be nested and condensed to generate a higher level superelement. Finally, after the DOF’s condensations with four levels of nested superelements, the whole panel structural elements will be condensed into the highest one with the master DOFs only at the traffic loading positions and boundary supports, as shown in Figure 4-5(f).

Based on this method, a numerical 3D finite element model is developed using the ANSYS 11.0 commercial software, where the corresponding material properties of the GFRP panels are listed in Table 4-1. Figures 4-6(a) to 4-6(e) show the numerically predicated results of the live load distribution factors for the first five HS-20 traffic load cases from the bridge’s one side to the middle, and the other five loadings should provide similar results due to the symmetries of the forces and structures in the numerical study. The field tested results reported by Zhang and Cai (2007) for this bridge are also digitized and plotted in the corresponding figures for comparisons. It can be observed that a good agreement is shown between the prediction and the field measurement which verifies the proposed modeling method. It is also noteworthy of being mentioned that the computing time for this specific project, using the same office computer with standard configurations, will be no more than 24 hours for all ten loading cases if using the sub-structuring method; while it will take about 48 hours running for only one case when using the traditional method and modeling the actual configurations directly.
**Fig. 4-5** Sub-structuring Modeling Procedures for the GFRP Panel Bridge under HS-20 Loads

| Table 4-1 Properties of GFRP Panel Bridge (Oghumu 2005) |
|-----------------|----------------|----------------|----------------|----------------|
| Element         | $E_x$  | $E_y$  | $G_{xy}$ | CTEX       | CTEY          |
| Unit            | Gpa    | Gpa    | Gpa      | L/L/$^\circ$C $\times 10^{-6}$ | L/L/$^\circ$C $\times 10^{-6}$ |
| GFRP Face       | 20.15  | 12.87  | 3.76     | 12.3        | 19.7          |
| GFRP Core       | 12.65  | 12.65  | 4.54     | 20.8        | 20.8          |
| Steel Beam      | 210    | 210    | 80       | 11.7        | 11.7          |
Fig. 4-6 Lateral Distribution Factor Results from ANSYS Predicated and Field Tests

4.2.3 Temperature Load Results

Kansas DOT measured the temperatures at the bridge’s top surface, bottom surface, and the ambient every two hours from December 2002 to July 2004 (Meggers 2006). In this section, the
measured temperatures during the period from January 24 to July 13 in 2004, shown in Figure 4-7, are chosen and used in the investigation of the thermal performances.

The thermal gradients, often inducing the bending deformations and self-equilibrating stresses, refer to the daily temperature variations along a bridge’s depth. The AASHTO LRFD (2007) design code specifies the multi-linear distribution patterns for traditional concrete and steel bridges while no such specifications are available for FRP bridges. In this sense, based on the measured temperatures at GFRP panel’s top and bottom surfaces, the temperature gradients through the depth are assumed in this study. According to the previous studies (Kong and Cai 2012a and 2012b), together with the consideration of the GFRP panel’s shallower depths and steel girders’ higher thermal conductivities, a linear temperature distribution is assumed through the depth of the sandwich panel and a constant distribution along the steel girder. Therefore, from the measured temperature results of the panels’ top and bottom surfaces, as listed in Table 4-2, the highest positive temperature differences is observed on June 23 at 11:50 during the hottest week from June 21 to June 27; and the maximum negative temperature differences happened on February 7 at 3:50 during the coldest week from February 7 to February 13. The temperature gradients adopted in analysis are shown in Figure 4-8.

Fig. 4-7 Measured Bridge and Ambient Temperatures from January 24 to July 13, 2004

The temperature gradients, often inducing the bending deformations and self-equilibrating stresses, refer to the daily temperature variations along a bridge’s depth. The AASHTO LRFD (2007) design code specifies the multi-linear distribution patterns for traditional concrete and steel bridges while no such specifications are available for FRP bridges. In this sense, based on the measured temperatures at GFRP panel’s top and bottom surfaces, the temperature gradients through the depth are assumed in this study. According to the previous studies (Kong and Cai 2012a and 2012b), together with the consideration of the GFRP panel’s shallower depths and steel girders’ higher thermal conductivities, a linear temperature distribution is assumed through the depth of the sandwich panel and a constant distribution along the steel girder. Therefore, from the measured temperature results of the panels’ top and bottom surfaces, as listed in Table 4-2, the highest positive temperature differences is observed on June 23 at 11:50 during the hottest week from June 21 to June 27; and the maximum negative temperature differences happened on February 7 at 3:50 during the coldest week from February 7 to February 13. The temperature gradients adopted in analysis are shown in Figure 4-8.
The uniform temperature variations, otherwise, often inducing the expansion and contraction movements of a bridge, are defined as the differences between the bridges’ maximum or minimum temperatures and the reference temperatures. The former one, also called the effective bridge temperature, is the average temperature along the cross section; while the later one, referring to conditions for bridges at zero stresses, is assumed as 4°C (40°F) in this study. Then, for the maximum bridge’s average temperature, it usually appears around the mid-day or the early afternoon during a hot day when the top surfaces are significantly heated by the solar radiation but the convection behaviors between the bridge surfaces and the air are weak; thus, the temperature measured on July 12 is used. For the minimum bridge temperature, it should be appearing at a cold day with a lower air temperature during the night without the solar radiation. Then, the measured temperature on January 29, with little temperature differences between the bridge and the air, is adopted. The two uniform temperature variation cases are also listed in Table 4-2.

Therefore, the bridge responses under the dead load, HS-20 traffic loads, and temperature variations are simulated. The results of the mid-span vertical deflections, right-end horizontal movements, and top and bottom surface stresses of steel girders at the mid-span, listed in Table 4-3, are discussed in this section. Additionally, these simulated mid-span deflections, stresses, and end displacements in Table 4-3 are normalized with the correspondence to the result of the HS-20 case, the material’s yielding strength, i.e., f_{steel} = 344Mpa (50ksi), and the result of the temperature increase case, respectively, and are shown in Table 4-4. It should be noted that, the measured and selected temperature loadings still may not definitely represent the most critical conditions. Some of the conclusions are drawn as follows.
Table 4-2 Measured Temperature Loads for Numerical Modeling

<table>
<thead>
<tr>
<th>Date and Time</th>
<th>Top (°F)</th>
<th>Bottom (°F)</th>
<th>Ambient (°F)</th>
<th>Average (°F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6/23/2004 11:50</td>
<td>146.4</td>
<td>88.7</td>
<td>82.4</td>
<td>117.5</td>
</tr>
<tr>
<td>2/7/2004 3:50</td>
<td>-11.2</td>
<td>15.8</td>
<td>4.1</td>
<td>2.3</td>
</tr>
<tr>
<td>7/12/2004 13:50</td>
<td>159.8</td>
<td>105.8</td>
<td>95.9</td>
<td>132.8</td>
</tr>
</tbody>
</table>

Table 4-3 Absolute Results for the GFRP Panel Bridge

<table>
<thead>
<tr>
<th>Loads</th>
<th>Middle Span</th>
<th>Support End</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Disp. z (mm)</td>
<td>Bot (Mpa)</td>
</tr>
<tr>
<td>Dead Load</td>
<td>-3.40</td>
<td>10.99</td>
</tr>
<tr>
<td>HS-20 Live Load</td>
<td>-6.58</td>
<td>21.49</td>
</tr>
<tr>
<td>Temp. Neg. Grad.</td>
<td>2.18</td>
<td>0.82</td>
</tr>
<tr>
<td>Temp. Uni. Dec.</td>
<td>-1.60</td>
<td>1.65</td>
</tr>
<tr>
<td>Temp. Uni. Inc.</td>
<td>5.16</td>
<td>-5.34</td>
</tr>
</tbody>
</table>

Table 4-4 Normalized Results for the GFRP Panel Bridge

<table>
<thead>
<tr>
<th>Loads</th>
<th>Middle Span</th>
<th>Support End</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Disp. z (L/L)</td>
<td>Bot (%)</td>
</tr>
<tr>
<td>Dead Load</td>
<td>0.52</td>
<td>3.19</td>
</tr>
<tr>
<td>HS-20 Live Load</td>
<td>1.00</td>
<td>6.23</td>
</tr>
<tr>
<td>Temp. Pos. Grad.</td>
<td>-5.37</td>
<td>-9.20</td>
</tr>
<tr>
<td>Temp. Neg. Grad.</td>
<td>-0.33</td>
<td>0.24</td>
</tr>
<tr>
<td>Temp. Uni. Dec.</td>
<td>0.24</td>
<td>0.48</td>
</tr>
<tr>
<td>Temp. Uni. Inc.</td>
<td>-0.78</td>
<td>-1.55</td>
</tr>
</tbody>
</table>

1. As is expected, the concentrated or distributed vertical loads, such as the self-weight and HS-20 live loads adopted in this study, produce the large responses at the mid-span but insignificant ones near the simply supported boundaries, as listed in Table 4-3. However, for temperature loadings, particularly for both of the positive thermal gradient and uniform temperature increase loading cases, the induced effects, as listed in Table 4-3, can no longer be ignored at the supports, where the large thermal stresses are generated due to the nonlinear thermal gradients and the different thermal coefficients of expansions between materials.

2. For the bridge’s vertical deformations induced by the live loads, the AASHTO LRFD (2007) requires that the calculated deflections, if using a live load distribution factor for this project, do not exceed the values of span-length/800, i.e., 17.145mm (0.675in), or span-
length/1000, i.e., 13.716mm (0.54 in), for pedestrian sidewalks, respectively. Since the induced bridge deflection under the HS-20 truck loadings after slab replacements is 6.58mm (0.26 in) as listed in Table 4-3, it is still satisfying the requirements.

3. The bridges’ vertical deflections induced from temperature effects are complicated. The large positive thermal gradients, causing the extensions of fibers through the depth of the superstructure, will induce the hogging actions, as the positive values listed in Table 4-3; while for the negative thermal gradients, with the contractions of slabs but the extensions of beams, the bridge behaviors will actually depend on the temperature values and the thermal expansion abilities of both the slabs and beams. In this study, the whole bridge performance is primarily dominated by the hogging behaviors of the steel girders due to their large cross section, even though these upwards bending behaviors are weakened by the sagging movements from the contraction tendencies of the GFRP panels.

4. Besides the thermal gradients, the uniform temperature variations also produce the vertical deflections, and this scenario is mainly originated from the different material thermal expansion coefficients at the interfaces between the GFRP panel and the steel girder. It can be observed that the induced bridge deflections in the uniform temperature increase case, listed in Table 4-3, are even comparable to that of the live load effects.

5. Table 4-4 shows the normalized results which are used to compare the bridge responses between different loading cases. First, the stress levels for this specific project, even though varied between each other, are generally taking a small proportion of the material strength capacity. Second, due to the short span and light weight of the current GFRP panels, the effects from the dead load are smaller than that of the live load. This scenario verifies one of the advantages that it will be benefit from a reduction in dead load and subsequent an increase in live load ratings after the slab replacements with FRP panels. Third, more attentions should be given to the temperature induced effects, since the calculated thermal stresses at the steel girders due to the large positive thermal gradients and uniform temperature increases, yet may not be the worst cases, are already exceeding to that of the live load and self-weight effects.

4.3 Parametric Study

A more general condition is further investigated by a parametric study in this section, where the thermal behaviors of a concrete bridge with AASHTO type I beams, shown in Figure 4-9, and a steel bridge with AISC W21×68 girders, shown in Figure 4-10, before and after slabs
replaced with GFRP panels, are considered. The corresponding beam and girder geometric information are plotted in Figure 4-11 and listed in Table 4-5 and Table 4-6, respectively. In addition, an identical concrete slab is assumed as the original deck for both bridge cases and the previously discussed GFRP panel is again adopted as the replacing panels in this study. For the convenience of comparisons, the characteristic of the concrete deck is artificially defined with a 10.16cm (4in) height and owing an equivalent rigidity to that of the GFRP panel, though most slabs are from 20cm to 25.4cm (8in to 10in).

![Concrete Beam Bridge](image1)

**Fig. 4-9** Concrete Beam Bridge

![Steel Girder Bridge](image2)

**Fig. 4-10** Steel Girder Bridge

![Concrete and Steel Girders Geometry](image3)

**Fig. 4-11** Concrete and Steel Girders Geometry

| Table 4-5 AASHTO Concrete Girders Section Properties (cm) |
|-----------------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|
| **Type** | **H₁** | **H₂** | **H₃** | **H₄** | **H₅** | **H₆** | **W₁** | **W₂** | **W₃** | **W₄** | **W₅** |
| I     | 71.1    | 12.7    | 12.7   | 27.9    | 7.6     | 10.16   | 40.6    | 30.5    | 15.2    | 12.7    | 7.6     |
Moreover, the loads that are considered in this parametric study still include dead loads, HS-20 live loads, and temperature effects. Specifically, the uniform and gradient temperature loads are referred to the temperature design criteria stipulated in the AASHTO LRFD (2007) as: (1) for the uniform temperature variations, considering the material types and bridge locations, a 44.4°C (80°F) temperature increase condition is assumed for all concrete, steel, and GFRP panel cases in this study; (2) for the thermal gradients, four solar radiation zones are divided in the United States and the corresponding positive and negative temperature distribution patterns are stipulated according to the bridge locations, material types, superstructure depths, and wearing surface properties. Thus, in this study, the original concrete beam and steel girder bridges before slab replacements, assumed being located in Kansas State, are conforming to the available temperature design codes AASHTO LRFD (2007) specified for this area; (3) for the GFRP panel bridges after slab replacements, however, no available temperature specifications are provided and some reasonable assumptions are made in this study. Based on the field monitoring and other analytical study experience as reported by Kong and Cai (2012a, 2012b) that, the GFRP panel bridge is often observed with (1) a relatively higher top surface temperature due to the material’s larger solar absorptivity; (2) an approximately linear distribution through the panel depth due to the lower thermal conductivity and hollowed section configurations; and (3) a constant or negligible gradient through the steel or concrete girders. Thus, a relatively higher temperature compared with that at concrete top surfaces, 21°C (70°F), are assigned at the GFRP slab’s top surface and a uniform distribution is assumed through the girder depths. All the temperature gradient distributions are shown in Figure 4-12 and Figure 4-13.

Therefore, the numerical models are established for all the parametric cases and the corresponding results in terms of the bridge vertical deflections, horizontal movements at bridge ends, and girder bending stresses are illustrated from Tables 4-7 to 4-14 for the convenience of readers, in which both the absolute results and the normalized results are listed. The normalized values are again calculated by dividing the bridge vertical deflections, girder stresses, and
horizontal movements with the correspondence to the HS-20 live loads case results, the material strength, i.e., $f_{\text{steel}} = 344\text{Mpa (50ksi)}$ and $f_{\text{concrete}} = 25\text{Mpa (3600psi)}$, and the temperature increases case results, respectively. Figures 4-14 to 4-21 show the comparisons of behaviors before and after slab replacements for concrete beam and steel girder bridges. A consistent legend convention is defined here and will be adopted through the following discussions, where the two characters in an ordinal sequence, denote the material types of the slab and beam, respectively, e.g., GC referring to the case with a GFRP (G) slab and a concrete beam (C).

![Fig. 4-12 Temperature Gradients for Concrete and GFRP Slab Bridges with Concrete Beams](image1)

![Fig. 4-13 Temperature Gradients for Concrete and GFRP Slab Bridges with Steel Girders](image2)

### Table 4-7 Concrete Slab and Concrete Beam Cases Absolute Results

<table>
<thead>
<tr>
<th>Loads</th>
<th>Middle Span</th>
<th>Support End</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Disp. z (mm)</td>
<td>Bot (Mpa)</td>
</tr>
<tr>
<td>Dead Load</td>
<td>-6.32</td>
<td>3.75</td>
</tr>
<tr>
<td>HS-20 Load</td>
<td>-3.20</td>
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<tr>
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<td>Temp. Uni. Inc.</td>
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### Table 4-8 Concrete Slab and Concrete Beam Cases Normalized Results

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<th>Middle Span</th>
<th>Support End</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Disp. z (L/L)</td>
<td>Bot (%)</td>
</tr>
<tr>
<td>Dead Load</td>
<td>1.98</td>
<td>15.11</td>
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<tr>
<td>HS-20 Load</td>
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<td>Temp. Pos. Grad.</td>
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<td>-5.06</td>
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### Table 4-9 GFRP Slab and Concrete Beam Cases Absolute Results

<table>
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</tr>
</thead>
<tbody>
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<td>Disp. z (mm)</td>
<td>Bot (Mpa)</td>
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<tr>
<td>Dead Load</td>
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<td>2.89</td>
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<tr>
<td>HS-20 Load</td>
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<td>Temp. Pos. Grad.</td>
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<td>Temp. Uni. Inc.</td>
<td>5.74</td>
<td>-1.31</td>
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### Table 4-10 GFRP Slab and Concrete Beam Cases Normalized Results

<table>
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<tbody>
<tr>
<td></td>
<td>Disp. z (L/L)</td>
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<tr>
<td>Dead Load</td>
<td>1.18</td>
<td>11.66</td>
</tr>
<tr>
<td>HS-20 Load</td>
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<td>9.67</td>
</tr>
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<td>Temp. Pos. Grad.</td>
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<tr>
<td>Temp. Uni. Inc.</td>
<td>-1.15</td>
<td>-5.26</td>
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### Table 4-11 Concrete Slab and Steel Girder Cases Absolute Results

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<th>Middle Span</th>
<th>Support End</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Disp. z (mm)</td>
<td>Bot (Mpa)</td>
</tr>
<tr>
<td>Dead Load</td>
<td>-5.72</td>
<td>21.58</td>
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<tr>
<td>HS-20 Load</td>
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<td>Temp. Uni. Inc.</td>
<td>-1.14</td>
<td>1.16</td>
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### Table 4-12 Concrete Slab and Steel Girder Cases Normalized Results

<table>
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<th>Loads</th>
<th>Middle Span</th>
<th>Support End</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Disp. z (L/L)</td>
<td>Bot (%)</td>
</tr>
<tr>
<td>Dead Load</td>
<td>1.25</td>
<td>6.26</td>
</tr>
<tr>
<td>HS-20 Load</td>
<td>1.00</td>
<td>5.37</td>
</tr>
<tr>
<td>Temp. Pos. Grad.</td>
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<td>-1.28</td>
</tr>
<tr>
<td>Temp. Uni. Inc.</td>
<td>0.25</td>
<td>0.34</td>
</tr>
</tbody>
</table>
Table 4-13 GFRP Slab and Steel Girder Cases Absolute Results

<table>
<thead>
<tr>
<th>Loads</th>
<th>Middle Span</th>
<th>Support End</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Disp. z (mm)</td>
<td>Bot (Mpa)</td>
</tr>
<tr>
<td>Dead Load</td>
<td>-3.40</td>
<td>10.99</td>
</tr>
<tr>
<td>HS-20 Load</td>
<td>-6.58</td>
<td>21.49</td>
</tr>
<tr>
<td>Temp. Pos. Grad.</td>
<td>7.04</td>
<td>-10.76</td>
</tr>
<tr>
<td>Temp. Uni. Inc.</td>
<td>4.45</td>
<td>-4.60</td>
</tr>
</tbody>
</table>

Table 4-14 GFRP Slab and Steel Girder Cases Normalized Results

<table>
<thead>
<tr>
<th>Loads</th>
<th>Middle Span</th>
<th>Support End</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Disp. z (L/L)</td>
<td>Bot (%)</td>
</tr>
<tr>
<td>Dead Load</td>
<td>0.52</td>
<td>3.19</td>
</tr>
<tr>
<td>HS-20 Load</td>
<td>1.00</td>
<td>6.23</td>
</tr>
<tr>
<td>Temp. Pos. Grad.</td>
<td>-1.07</td>
<td>-3.12</td>
</tr>
<tr>
<td>Temp. Uni. Inc.</td>
<td>-0.68</td>
<td>-1.34</td>
</tr>
</tbody>
</table>

Figure 4-14 and Figure 4-15 show the results of the vertical deflections for the steel and concrete bridges, respectively. For the live loads induced responses, they should have remained unchanged before and after slab replacements since an equivalent bridge rigidity was artificially defined during the modeling. However, the discrepancies observed from the results, under the HS-20 load cases, are attributed to the connection behaviors between the slab and beam. Full composite actions are assumed between the concrete slabs with the steel girders and concrete beams, while this assumption may be inappropriate for the GFRP slab bridges. In practice, it is actually difficult to construct fully rigid connections between the GFRP panel with the concrete beams and steel girders; and a routine connecting method is often implemented by bonding with epoxy materials or clamping with special designed devices. Thus, the partial composite actions assumed and modeled in the study induce relatively lower bridge rigidities and larger deflections after slab replacements.

In addition, the thermal gradient effects on both the concrete beam and steel girder cases should be given due attentions since the deflections are evidently increased after slab replacements. These deflections, yet not the worst cases, are comparable or exceeding to the responses from that of the live and dead loads. Importantly, the uniform temperature variations
may be insignificant for the original CS and CC cases, where almost none responses appeared. However, the distinctively different thermal expansion coefficients between the GFRP and concrete or steel materials are inducing evident increases of deflections after slab replacements.

![Vertical Deflections of Steel Girder Bridge](image1)

**Fig. 4-14** Vertical Deflections of Steel Girder Bridge

![Vertical Deflections of Concrete Beam Bridge](image2)

**Fig. 4-15** Vertical Deflections of Concrete Beam Bridge

Figure 4-16 and Figure 4-17 show the stresses induced at girder’s bottom surfaces for both steel and concrete beam bridges, respectively. It can be observed that, the temperatures induced behaviors, either from the gradient or the uniform temperature increase case, have little variations after the slab replacements. This scenario is resulted from the linear or zero temperature gradients along the beam depths, since only nonlinear thermal gradients can provide thermal stresses.
However, for the induce stresses at beam’s top surfaces, shown in Figure 4-18 and Figure 4-19, respectively, the apparent variations are again observed after the slab replacements. Similar to the previous observations of the deflection performances, the combined effects of the nonlinear thermal gradients and different material thermal expansion coefficients can generate considerably larger thermal stresses than other loading cases.

**Fig. 4-16** Steel Girder Bottom Surface Stresses of Steel Bridges

**Fig. 4-17** Concrete Beam Bottom Surface Stresses of Concrete Bridges
Figure 4-18 Steel Girder Top Surface Stresses of Steel Bridges

Figure 4-19 Concrete Beam Top Surface Stresses of Concrete Bridges

Figure 4-20 and Figure 4-21 show the results of the horizontal movements at the bridge ends after slabs replacement, respectively. It can be clearly observed that the uniform temperature variations still have the dominant effects. It should be noted that for the slab on girder bridges, as being studied in this study, the horizontal movements are largely determined by the deformation of the girders while the GFRP panel make little contributions due to its small geometries. It also explains the behaviors that, even though the GFRP panel owing much different expansion coefficients, the induced horizontal movements only have little changes after the slab replacements for both steel girder and concrete beam cases.
4.4 Conclusion

In this research, the thermal behaviors of a GFRP panel bridge are firstly investigated based on a field live load test and a long-term monitoring program. Then, the two general concrete beam and steel girder bridges are adopted to compare their performances, before and after slab replacements, through a parametric study. Some of the important observations are summarized as follows:

1. The proposed sub-structuring modeling method is valid. This method is proven to be particularly efficient for modeling the complex GFRP honeycomb hollow section sandwich panel bridges under live loads and through-depth thermal loads.
2. For the slab replacement project conducted by KSDOT, the measured nonlinear thermal gradients and uniform temperature variations from January 24 to July 13, 2004, yet not the worst cases, are already generating larger effects compared with the dead load and HS-20 live load effects, even though the deflections are still within the AASHTO LRFD (2007) requirements and the stresses are only taking a small portion of the materials’ strength capacity.

3. Thermal gradients will induce evident vertical deflections even for simply supported bridges. By replacing concrete slabs with GFRP panels, the produced deflections are increased compared with the original concrete slab cases. These deflections may even be larger than that caused by the live load and dead load effects for the same replacement case. Additionally, different from the traditional concrete and steel bridges, where vertical movements can hardly be generated under the uniform temperature variations, the GFRP panel bridges could provide large movements due to the distinctive material thermal properties at the interfacial locations between GFRP panel and the concrete or steel beams.

4. Thermal stresses are induced due to the nonlinear thermal gradients and the material property differences at the interfaces between two different materials. It has been observed that, by replacing concrete slabs with GFRP panels, the induced thermal stresses are evidently increased especially at the top surfaces of the girders. This phenomenon should be carefully considered, since the thermal stresses are already comparable and even exceeding to the effects from that of the dead load and live loads.

5. Bridge horizontal behaviors are still primarily determined by the uniform temperature variations. For slab on girder bridges, even though the GFRP panel has distinctive thermal expansion coefficients, yet the induced horizontal movements, after slab replacements, are still mainly contributed by the steel girders or concrete beams since the GFRP panel has small geometric sizes. However, for pure slab bridge systems where girders are not a part of the system, the responses between the FRP slab bridge and the concrete slab bridge may be more different.

4.5 References


Meggers, D. A. (2006). Personal communication, Kansas Department of Transportation


CHAPTER 5. FIELD MONITORING STUDY OF AN INTEGRAL ABUTMENT BRIDGE SUPPORTED BY PRESTRESSED PRECAST CONCRETE PILES ON SOFT SOILS - PART A

5.1 Introduction

Expansion and contraction are two unavoidable responses of bridges caused by the variation of temperatures, creep and shrinkage of materials, etc. Traditionally, a relief system, consisting of expansion joints, bearing supports, or other devices, are provided to accommodate such movements. After years of services, however, this relief system, especially the expansion joints, is considered to be one of the most vulnerable elements affecting the sustainability of bridges. For example, as reported by Mistry (2005) and Thippeswamy et al. (2002), water or deicing chemicals, leaking through the joint gaps onto the underlying structures, could lead to the steel deterioration and concrete spalling. In addition, the expansion joints are often subjected to some cycling and devastating loadings, e.g., the extreme daily or seasonal thermal variations, freeze-thaw cycles, overloads, ice-breaking equipment impacts, etc. Thus, the joints tend to be impaired, and the damages could be aggregated if the movements are obstructed by the debris or dirt. Last but not the least, the applications of expansion joints would yield huge life-cycle expenditures, from the beginning of the construction through the whole service life, involving the expense on the design, purchase, installation, replacement, maintenance, and also the extra spending on the retrofits and rehabilitations of other damaged structure elements.

In light of these negative aspects, researchers have always been trying to eliminate expansion joints wherever possible, and the concept of integral bridges without joints was inspired as a result. A full integral abutment bridge (IAB), as shown in Figure 5-1, refers to a single or multi-span bridge which has its superstructure, i.e., concrete slabs, prestressed concrete beams, steel girders, and approach slabs, cast monolithically with the stub type abutment, and founded on a single row of piles. In such a configuration, the joint issues can be resolved. The horizontal movements from the superstructure, however, are transferred to the substructure and accommodated by the complicated soil-structure interaction behaviors.
The benefits of the IABs have been widely accepted during the past several decades. A survey conducted by the Federal Highway Administration (FHWA) on the current practice of IABs from the fifty state Departments of Transportation (DOTs), District of Columbia DOT, Puerto Rico Highway and Transportation Authority, and Federal Lands Highway Division, shows that the number of the designed and built of IABs has increased significantly over 200% from the year of 1995 to 2004, and over 90% states have a policy to construct jointless bridges whenever possible (Maruri and Petro 2005). Though having been accepted, the IABs have not been widely applied in practice, and the current designs and constructions are mostly relying on empirical practice. Thus, it is difficult to further extend the benefits of IABs and apply them on other more complicated soil conditions and structural configurations.

The field monitoring method is commonly adopted in studies to clear all the uncertainties of IABs. Using this approach, the following design and construction assumptions are investigated and justified: (a) the maximum allowable design criteria (e.g., total and individual bridge span lengths and skews); (b) the structure design parameters (e.g., types and orientations of the pile, abutment, and wingwall); (c) the soil-structural interaction behaviors (e.g., between the soil-pile, abutment-backfill, and approach slab-backfill); (d) the joint connection effects (e.g., at the interfacial locations between the abutment-deck-girder, abutment-pile cap, approach slab-backfill, and intermediate pier-girder); (e) the stress relief mechanisms (e.g., diameters, depths, and filling materials of the pre-sized holes surrounding the piles, and the compacting degree of the backfill materials behind abutments); and (g) the long term effects (e.g., the thermal, shrinkage, creep, and steel relaxation) (Dunker and Liu 2007; Arockiasamy et al. 2004). Specifically, some of the recent monitoring programs conducted from the year of 2004 to 2010,
listed in Table 5-1, summarize most of the concerning items on the behaviors of superstructure and substructure in the aspects of the strains and stresses, deformations, and environmental conditions.

In the state of Louisiana, there are no full IABs and only semi-IABs have ever been constructed in the past. In this case, the Louisiana State Department of Transportation and Development (LADOTD) designs the first two full IABs on soft and stiff soils conditions, respectively, with the purpose to investigate their behaviors and to provide a reliable reference for their future constructions under the Louisiana’s environmental and soil conditions. Field monitoring programs were conducted on both of these two bridges. This paper presents the results for one of these two bridges, Caminada Bay Bridge on the soft soil condition, over one year since August, 2011. The following discussions are primarily emphasized on the bridge behaviors due to the daily and seasonal temperature variations, i.e., temperature distributions, abutment rotations and displacements, slab positive and negative bending strains, pile strains, and backfill pressures.

**Table 5-1** Lists of the Field Monitoring Programs Reported in the Literature

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Strain &amp; Stress</th>
<th>Deformation</th>
<th>Environment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge 55555, MN</td>
<td>concrete girder; deck; backfill pressure; steel rebar at connection details; pile curvature and axial stress</td>
<td>girder deflection; abutment pier rotation; bridge expansion and contraction; abutment movement</td>
<td>thermistor in gages; thermal gradient; air temperature; solar radiation; relative humidity</td>
</tr>
<tr>
<td>Huang et al. (2004)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Guthrie County &amp; Story County, IA</td>
<td>pile; PC girder; pile cap</td>
<td>abutment longitudinal &amp; transverse displacement; differential displacement between pile-pile cap, girder-pier, girder-backwall; pile cap rotation</td>
<td>thermistor in gages; thermal gradient; air temperature</td>
</tr>
<tr>
<td>Abendroth and Greimann (2005)</td>
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### Table 5-1 Lists of the Field Monitoring Programs Reported in the Literature (Continued)

<table>
<thead>
<tr>
<th>Location</th>
<th>Monitored Elements</th>
<th>Measured Parameters</th>
</tr>
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<tbody>
<tr>
<td>SR249; I65; SR18, IN</td>
<td>pile; abutment rotation; relative displacement of girder-abutment and pier-girder;</td>
<td>thermistor in gages;</td>
</tr>
<tr>
<td>Frosch et al. (2006)</td>
<td>pier movement and rotation,</td>
<td>Holland Sanatorium, PA;</td>
</tr>
<tr>
<td></td>
<td>abutment and stringer rotation; relief slab displacement</td>
<td>abutment rotation, bpm, thermistor in gages;</td>
</tr>
<tr>
<td>Scotch Road, NJ</td>
<td>pile; abutment; MSE wall; galvanized sleeve;</td>
<td>temperature in abutment and deck;</td>
</tr>
<tr>
<td>Hassiotis et al. (2006)</td>
<td>abutment and stringer rotation; relief slab displacement</td>
<td>Holland Sanatorium, PA;</td>
</tr>
<tr>
<td>Bridge No. 109, 203, 211, 222, PA</td>
<td>pile; backwall; girder; approach slab reinforcing bar;</td>
<td>ambient temperature; relative humidity; air pressure; rainfall; solar radiation; wind speed; wind direction;</td>
</tr>
<tr>
<td>Laman et al. (2006)</td>
<td>abutment and backwall displacement; abutment rotation,</td>
<td>Holland Sanatorium, PA;</td>
</tr>
<tr>
<td></td>
<td>bridge longitudinal and transverse displacement; abutment rotation,</td>
<td>Holland Sanatorium, PA;</td>
</tr>
<tr>
<td>OW Bridge, MA</td>
<td>backfill pressure; pile;</td>
<td>thermistor in gages;</td>
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<tr>
<td>Brena et al. (2007)</td>
<td>abutment and stringer rotation; relief slab displacement</td>
<td>Holland Sanatorium, PA;</td>
</tr>
<tr>
<td></td>
<td>shape of concrete deck; approach embankment settlement; EPS layer thickness</td>
<td>Holland Sanatorium, PA;</td>
</tr>
<tr>
<td>Blue Spring Run Bridge, VA</td>
<td>pile; backwall</td>
<td>Holland Sanatorium, PA;</td>
</tr>
<tr>
<td>Hoppe and Bagnall (2008)</td>
<td>abutment drilled shaft rotation; concrete deck shrinkage and creep</td>
<td>Holland Sanatorium, PA;</td>
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<tr>
<td>Kii Bridge, HI</td>
<td>abutment drilled shaft rotation; concrete deck shrinkage and creep</td>
<td>Holland Sanatorium, PA;</td>
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<td>Ooi et al. (2010)</td>
<td>abutment drilled shaft rotation; concrete deck shrinkage and creep</td>
<td>Holland Sanatorium, PA;</td>
</tr>
<tr>
<td></td>
<td>concrete topping precast planks; abutment wall; drilled shafts; backfill pressure;</td>
<td>Holland Sanatorium, PA;</td>
</tr>
<tr>
<td>Nash Stream, ME</td>
<td>pile; abutment;</td>
<td>thermistor in gages;</td>
</tr>
<tr>
<td>Davids et al. (2010)</td>
<td>abutment drilled shaft rotation; concrete deck shrinkage and creep</td>
<td>Holland Sanatorium, PA;</td>
</tr>
<tr>
<td></td>
<td>pile and abutment movement; pile displacement</td>
<td>Holland Sanatorium, PA;</td>
</tr>
<tr>
<td></td>
<td>Thermistor in gages;</td>
<td>Holland Sanatorium, PA;</td>
</tr>
</tbody>
</table>

#### 5.2 Bridge Descriptions

The Caminada Bay Bridge, shown in Figure 5-2, is located at Grand Isle, LA (29°15'48" N 89°57'24" W), about 160km (100 miles) to the south of New Orleans, LA. The old jointed
bridge was demolished, and a new full IAB was built next to it. The total length of the bridge is 1202m (3945 ft), while the monitoring program is conducted on the first 11 spans, shown in Figure 5-3, including a 3m (10 ft) sleeper slab, a 12m (40 ft) approach slab, a 91m (300 ft) continuous concrete slab, and the substructure underneath, i.e., the abutment, pile, and soil. The width of the bridge is 15m (50 ft) consisting of two 6.4m (21 ft) lanes and a 2m (7 ft) sidewalk on the northern side. The slabs of the bridge are fully integrated with the first bent (Bent1) at the left end, simply supported on the eleventh bent (Bent11) at the right end, and rigidly connected with all the other interior bents in between, where each bent is further rigidly supported on a row of four prestressed precast concrete (PPC) piles. The soils, referred to the boring log near Bent1, are approximately subdivided into two layers, including a medium sandy soil layer under the water level from the ground to the depth of 18.9m (62 ft) followed by a medium clay one.
5.3 Instrumentation

Bridge Diagnostics, Inc. (BDI) was contracted by Louisiana Transportation Research Center (LTRC) and Louisiana State University (LSU) to install the bridge monitoring system. In this project, a total of 81 instruments were applied on the Caminada Bay Bridge, listed in Table 5-2, including the vibrating wire strain gages, vibrating wire tiltmeters, vibrating borehole wire extensometers, vibrating wire pressure cells, piezometers, and vibrating wire thermistors. The large application of the vibrating wire gages is due to their good performance, without suffering from drifts, for the long-term monitoring; and also each sensor is provided with an extra temperature thermistor that the temperature of the element can be simultaneously obtained.

Table 5-2 Instrumentations Applied on Caminada Bay Bridge

<table>
<thead>
<tr>
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<th>Gages</th>
<th>Location</th>
<th>Numbers</th>
</tr>
</thead>
<tbody>
<tr>
<td>Superstructure</td>
<td>Sisterbar</td>
<td>Approach slab Bottom (embedded)</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Span 1,3, &amp; 5 Bottom (embedded)</td>
<td>2x3=6</td>
</tr>
<tr>
<td></td>
<td>Strain Gage</td>
<td>Bent 1,2, &amp; 5 Top (embedded)</td>
<td>2x3=6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Span 3, 4, 5, &amp; 6 Bottom (surface)</td>
<td>2x4=8</td>
</tr>
<tr>
<td>Substructure</td>
<td>Sisterbar</td>
<td>Two easternmost piles at Bent1</td>
<td>16x2=32</td>
</tr>
<tr>
<td></td>
<td>Tiltmeter</td>
<td>Bent1 and Bent11 (surfaces)</td>
<td>1x2=2</td>
</tr>
<tr>
<td></td>
<td>Thermistor</td>
<td>Bent1 (embedded)</td>
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</tr>
<tr>
<td></td>
<td>Pressure cell</td>
<td>Bent1 back face</td>
<td>9</td>
</tr>
<tr>
<td></td>
<td>Soil Strain Meter</td>
<td>Backfill behind Bent1</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Thermistor</td>
<td>Backfill behind Bent1</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Piezometers</td>
<td>Backfill behind Bent1</td>
<td>6</td>
</tr>
</tbody>
</table>

5.3.1 Superstructure Instrumentation

For the superstructure, a total of 22 sensors, with 14 embedded sisterbars and 8 surface strain gages, as shown in Figure 5-4, are applied on the 46 cm (18in) depth decks and used to measure both the positive and negative strains due to the temperature changes. Specifically, the embedded sisterbars were placed at the rebar locations before the pouring of concrete with 8cm (3in) above the slab bottom surfaces on the approach slab, Span1, Span3, and Span5, and 5cm (2in) below the top surface on the Bent1, Bent2, and Bent5. The surface strain gages, otherwise, are mounted under the slab bottom surfaces from Span3 to Span6 after the completion of the concrete pouring.
5.3.2 Substructure Instrumentation

For the substructure, a total of 59 instrumentations are installed as shown from Figures 5-5 to 5-7. Specifically, (a) 32 sisterbars, with four groups of four gages, were installed on the four corners of the two 24m (80ft) long PPC piles at the easternmost of Bent1 to measure the pile strains, where the distances from the sisterbars at each pile sections to the bottom surface of Bent1 are 1.2m (4ft), 3.7m (12ft), 6.1m (20ft), and 8.5m (28ft), respectively; (b) 2 tiltmeters were attached at the middle section of the 1.2m (4ft) high Bent1 and Bent11 facing towards the water to record the bent rotations; (c) two rows of 9 soil pressure gages, with one row placed 30cm (1ft) away from the slab bottom surfaces and the other at about the bent middle section, were mounted on the backwall of Bent1 to measure the soil pressures on the abutment, and the pressure gages in the transverse direction from the easternmost end of Bent1 are 1.4m (56in), 3.5m (138in), 5.6m (221in), 7.7m (303in), and 11.9m (468in), respectively; (d) 4 soil strain gages, with 2 connected to Bent1, and another 2 floating about 1.2m (4ft) and 2.1m (7ft) away from Bent1, were applied to investigate the soil deformation or Bent1 movement; and also (e) several other gages were embedded in Bent1 and through the surrounding soil depth to obtain the corresponding structure temperatures and pore water pressures.
5.3.3 Data Acquisition System

The data acquisition system, shown in Figures 5-8(a) and 5-8(b), consists of a CR1000 datalogger, AVW200 interface, AM16-32 multiplexer, wireless cell modem, and solar battery. In addition, it has been outfitted with a wireless communication link so that the operation and data collection can be handled remotely from a computer by running the LoggerNet program installed at LSU. Specially, the acquisition system has also been equipped with a battery back-up mechanism that will allow continuous data collection throughout a typical power outage, and alert emails will be sent to researchers when the system is working on battery.
5.3.4 Data Post-Processing

The data acquisition starts from 08/11/11 and continues over one year in which the raw data is recorded every 3 minutes 20 seconds. In addition, these raw data have been manipulated by the LoggerNet program and the corresponding average, maximum, and minimum results within each hour are provided. Two post-processing steps are conducted on the raw data before further data analysis. First, the outlier data has to be picked out and removed. Otherwise, they may affect the results in the subsequent calculations, especially for those cases when one, or several, data within one hour is extremely large or small. Secondly, complying with the requirements of the sensor suppliers, a proper temperature correction process should be performed for certain sensor types. That is due to the differences of the thermal expansion coefficients between the steel wire in the sensor gages and that of the measured elements, e.g., the strain gages in concrete slabs and PPC piles.

For the readings from all the 81 instrumentations, most of the outliers from the raw data are obvious and appearing within certain time intervals that can be easily filtered and removed by using the Excel tools; while several of the others occur randomly and dispersedly that the aforementioned method is no longer convenient. Hence, a program using the statistical theory is coded in MATLAB (2010) for the data post-processing. Generally, many algorithms have been proposed for the data outlier removal in the statistic field, such as the Standard Deviation Method, Z-score, Modified Z-score, Tukey’s, Adjusted Boxplot, MAD, Median Rule, etc. Each method has its own advantages and restrictions, and most of the methods are based on one assumption that the sample data is normally distributed. As is shown in Figure 5-9, about 68%, 95%, and 99.7% of the data will be falling within 1, 2, and 3 times the standard deviation from the mean if the data follows a normal distribution. In this sense, if any data existing with a distance, e.g., 2 or 3 times of the standard deviation away from the mean, it can be considered as an outlier.

![Normal Distribution Curve](image-url)

**Fig. 5-9 Normal Distribution Curve**
The data of a strain gage in the concrete Span5 slab, shown in Figure 5-4, is taken out as an example to illustrate and discuss the outlier removal algorithm. Four sets of the sample data within one hour at four representative days, i.e., 08/11 17:00, 09/11 24:00, 10/11 03:00, and 11/11 09:00, are randomly selected from the raw readings, and the normality test is conducted on these data to justify their normality. Figure 5-10 and Figure 5-11 show the representative normality test plots of the strain and temperature readings at 08/11 17:00, respectively. All the calculated results are listed in Table 5-3, where the conclusion, that the null hypothesis is rejected or the distribution is non-normal, can be drawn if the P-value is smaller than 0.01 or the A-squared value is larger than the critical values, i.e., 0.787 and 1.072 for 95% and 99%, respectively. Based on the study, the strain data within an hour can be considered as normally distributed in most cases; while for the temperature results, the data can be more uniformly distributed within an hour such as in the case of 08/11 17:00. In addition, for those of sensors embedded in the soil, e.g., strain gages embedded in the piles under the ground, the temperatures at those locations are almost constant; thus, the temperature data cannot be always considered as normally distributed. Therefore, through a comprehensive comparison between the different outlier methods, together with the consideration of the distribution patterns of the current readings, the SD and Tukey’s method are adopted and coded in MATLAB (2010), and the following discussions will be based on the data after the outlier removals and temperature corrections.

![Fig. 5-10 Normality Test of Span5 Strain Data at 08/11 17:00](image)

Fig. 5-10 Normality Test of Span5 Strain Data at 08/11 17:00
Table 5-3 Normality Test of the Strain and Temperature Readings of Strain Gage at Slab5

<table>
<thead>
<tr>
<th>Date</th>
<th>Strain Reading A-squared</th>
<th>Strain Reading P value</th>
<th>Temperature Reading A-squared</th>
<th>Temperature Reading P value</th>
</tr>
</thead>
<tbody>
<tr>
<td>08/11/11 17:00</td>
<td>0.122</td>
<td>0.984</td>
<td>17.627</td>
<td>0</td>
</tr>
<tr>
<td>09/11/11 24:00</td>
<td>0.231</td>
<td>0.77</td>
<td>0.714</td>
<td>0.051</td>
</tr>
<tr>
<td>10/11/11 03:00</td>
<td>0.213</td>
<td>0.827</td>
<td>0.346</td>
<td>0.442</td>
</tr>
<tr>
<td>11/11/11 09:00</td>
<td>0.437</td>
<td>0.264</td>
<td>0.638</td>
<td>0.081</td>
</tr>
</tbody>
</table>

5.4 Field Monitoring Results

During the monitoring period, from 08/11/11 to 08/11/12, the construction of the bridge had not been completed yet, and no traffic was passing on the bridge. Thus, the following discussions are mainly about the bridge performance due to the temperature changes. The initial reference condition, or the baseline, for all the instrumentations were set up on 08/11/11.

5.4.1 Environmental Conditions

Even though no instrumentations were specifically applied to measure the environmental conditions, e.g., the ambient temperatures and wind speeds, this information is still of great importance. On one hand, the major uncertainties of IABs are largely caused by the temperature changes within the bridges, and those variations in turn are literally determined by the environmental conditions. On the other hand, the current AASHTO LRFD (2007) design temperatures for the traditional jointed bridges, either the uniform or the gradient ones, are all specified based on the bridge’s local environment conditions, while the adaptabilities of these specifications on IABs deserve further investigations. In this sense, the environmental
temperatures and wind speeds, recorded at the bridge site from a nearest weather station located at Grand Isle, LA (http://tidesandcurrents.noaa.gov), are referred and discussed in this section.

Figure 5-12 shows the measured hourly-varying air temperatures and wind speeds at the weather station, and Figure 5-13 shows the daily and monthly average temperatures calculated from the hourly-varying ones. From these two plots, the periodical and cycling trends can be clearly observed for all three temperature variations, especially the sinusoidal pattern shown from the hourly-varying temperatures. In this sense, the temperatures of the bridge may no longer be measured by the thermistors, but rather be more conveniently and economically predicted based on the local environmental conditions. For example, they can be predicted either by fitting a sinusoidal function between the bridge and air temperatures, or by the numerical simulations taking the environments as boundary conditions. In addition, the minimum and maximum ambient temperatures during the monitoring period, as marked in Figure 12, are approximately 4.4°C (40°F) to 29.4°C (85°F), respectively, with a difference of 25°C (46°F). Based on the AASHTO LRFD (2007) specification, the bridge should be located at a moderate climate region since the number of the freezing days per year, the day with the temperature less than 0°C (32°F), are less than 14. Then, the corresponding temperature specifications for the moderate climate region are compared with the field measurements in the following discussions.

The variations of the wind speeds are also shown in Figure 5-12. It can be observed that the wind speeds and air temperatures generally do not show high correlations, with a correlation factor of -0.3 in this case. The occurrences of the peak wind speeds, however, are almost in coincidence with the conditions when the temperatures are at sudden drops. In addition, the measured speed values are generally wandering around 5 m/s, with some days larger than 10 m/s but smaller than 14 m/s. According to Elbadry and Ghali (1983), the wind speeds will determine the convection coefficients, and those coefficients in turn will affect the heat transfer mechanism at the surfaces of the bridge slabs. For example, in one of the commercial programs FEMMASSE (2000), which is capable of predicting the bridge deck temperatures, 5 m/s is a threshold for the calculation of the convection coefficients, and the corresponding values at 10 m/s is 2 times larger than that at 5 m/s (Schlangen 2000). In this sense, under the current air temperatures and wind speeds, the thermal responses at the top surfaces of the slabs should be expected with high variations.
Fig. 5-12 Measured Environmental Conditions at the Weather Station

Fig. 5-13 Hourly, Daily, and Monthly Varying Air Temperatures

5.4.2 Temperatures

The thermal performances of the bridges are directly determined by the variations of the temperatures within the bridges. These temperatures, however, are not uniformly distributed at various locations or within different structure components. In this section, with the help of the
largely applied instrumentations where each one being attached with a thermistor, the measured bridge temperatures in terms of the seasonal variations, daily gradients, and distributions from the slab top surface to the pile bottom section are observed and discussed.

Figure 5-14 shows the measured hourly-varying temperatures at the bent top surface, approach slab bottom surface, and deck slab bottom surface, respectively. It can be observed that the top surface of the slab has significant temperature variations, especially during the summer seasons; while during the winter season, these high variations decreases, and the temperatures at top and bottom surfaces are approaching to each other. The temperatures at the approach slab bottom surfaces, otherwise, are always in between to that of the bent top and slab bottom surfaces. These temperature distribution scenarios can be clearly explained by the heat transfer mechanisms. The high and varying top surface temperatures are primarily affected by the solar radiation and the top surface convection. After the top surface is heated, the high energy will transfer and conduct through the slab depth to the slab bottom surface. When arriving at the bottom surface of the deck slab, the high heat will be either blown away by the adjacent air through convection or transferred to the soil underneath through conduction.

For the seasonal and uniform slab temperatures during the monitoring period as indicated in Figure 5-14, they are approximately 42°C (108°F) and 6.7°C (44°F) at the bridge top and bottom surfaces, respectively, with a difference of 36°C (64°F). As is specified in the AASHTO LRFD (2007) code, two procedures are provided for the uniform design temperature calculations, i.e., (1) procedure A specifies the minimum and maximum values, -12°C (10°F) and 27°C (80°F), respectively, for the moderate climate region as this bridge is located, and the differences between the extended lower or upper boundary and the base construction temperature assumed in the design are used to calculate the thermal movement; (2) procedure B provides the extreme design temperature, 40°C (105°F) and 1.7°C (35°F), respectively, with a 38°C (70°F) difference. Therefore, if comparing with the AASHTO LRFD (2007) code, together with the consideration of the two measured extreme air temperatures, 4.4°C (40°F) and 29.4°C (85°F), the effective variations of the bridge temperature that will induce the seasonal movement for this IAB is 37°C (68°F), which is almost reaching the values of 38°C (70°F) per procedure B, and already exceeding the value per procedure A, 22°C (40°F), if assuming the base construction temperatures equivalent to the minimum air temperature of 4.4°C (40°F).
Besides the bridge uniform temperature variations, the gradient distributions are also important in design. For temperature distributions along and perpendicular to the traffic directions, the temperature differences are negligible based on the field measurements which are not demonstrated in this paper. For the vertical gradient through the slab depth, however, the differences are apparent. Figure 5-15 and Figure 5-16 show the temperatures measured at the top and bottom surfaces of the slab deck and that of the air during the hottest week, i.e., 08/19/11 to 08/26/12, and the coldest week, i.e., 12/29/11 to 01/26/12, respectively. During the hottest week, the bridge top surface and air temperature show a high correlation with almost the same variation trends, even though a certain lagging effect is observed between the temperatures at the top and bottom surfaces due to the thermal inertia for such a 46cm (18in) depth slab. During the coldest week, however, the temperature gradient is not as significant and all the top and bottom temperatures are closer.

According to AASHTO LRFD (2007), the positive and negative thermal gradients are specified based on the four subdivided radiation zones, together with the consideration of the superstructure materials, geometries, and overlays. Then, if following the code procedures, the positive temperature gradients of the slab are supposed to have a difference of 10.2°C (18.5°F), and multiplied by -0.3, with a -3.1°C (-5.6°F) negative one. In comparisons, the field measured positive and negative temperature gradients, appearing on 08/20/11 17:00 with a difference of
10°C (18°F) and on 1/6/12 07:00 with a value of -1.7°C (-3°F), respectively, are also almost reaching to that of the code specifications.

4. Besides the bridge slab temperatures, Figure 5-17 shows all the temperature variations from the top of the bridge to the bottom of the piles, and some of the observations are obtained as
follows. First, the solar radiation can only be transferred to the abutment bottom surfaces. Thus, the majority of both the daily and seasonal temperature variations are happening at the superstructure and they will ultimately induce the thermal responses of the bridges. Second, the temperatures in the soil are almost constant, even though they may be affected by the environments near the ground, such that the temperature in soils could be higher than that in the slabs and bents during the winter seasons. Based on the observations, therefore, the bridge temperature variations can be generally represented by that of the slabs. Figure 5-18 shows the best fitted relationships between the bridge and air temperatures, where the former one is assumed as the average between the readings from the sensors at the bent top and slab bottom surfaces. It can be observed that the bridge and air temperatures are almost linearly related in the middle temperature range, while polynomial curves shows better results in the lower or higher temperature ranges. This conclusion justifies the one of the arguments discussed above that it is advisable to predict the bridge temperature using the air temperature.

![Fig. 5-17 Measured Bridge Temperatures through Bridge Depth](image-url)
5.4.3 Slab Strains

For concrete slabs, the strains in the steel rebar and the surrounding concrete should be the same prior to the cracking of the concrete or yielding of the steel. Since the concrete is weak in tension, about 10% of its limit compressive strength, it may crack at the early stage of loadings. When in compression, however, the concrete can sustain up to 300 microstrain.

Figure 5-19 shows one representative strain gage’s readings at the Bent5 top rebar locations. The highly-varying strain variations can be observed, and the long term seasonal trends show a negative correlation with the temperature changes. In addition, the induced stresses during the temperature decrease of 18°C (64°F), if using the maximum 100 microstrain, would be about 2.48mpa (360psi). This value is about 9% of the concrete compressive strength. In this sense, the top surface of the bent has a high possibility of cracking due to the temperature variations. Similarly, another reading of the representative gage, embedded in the bottom parts of the Span5 slab, is shown in Figure 5-20. The induced compressive stress, if using 40 microstrains, is only about 2% of the compression strength.
Figure 5-21 shows the comparisons between the strains measured at the top surface of Bent2 and bottom surfaces of Span1. A good negative correlation behavior can be observed between them, and that can be attributed to the slab continuity over the bents due to the rigid connections. In addition, Figure 5-22 shows the slab strains distributed along the bridge length due to the temperature increase conditions with the correspondence to 08/11/11, i.e., about 5°C (9°F) increase on 08/14/11 and 27°C (49°F) increase on 1/13/12. It can be observed that no significant differences of strains appearing between the bridge end and middle span, even though
relatively larger strains should have been expected due to the abutment and soil restraints at the bridge end location. This scenario may partly be attributed to the fact that (a) the slab strains in this bridge are mostly induced by the temperature gradients rather than the seasonal variations; (b) the restraints from the soft soils at the bridge ends are not significantly strong; or (c) the rigid connecting behaviors between bents and slabs provide more evident structure rigidities than that from the backfills.

Fig. 5-21 Measured Strain at Span1 and Bent2

Fig. 5-22 Slab Strain Distribution w.r.t. Temperature Variations
5.4.4 Abutment Behaviors

The displacement and rotation of the abutments are measured through the soil strain meters and tiltmeters, where the positive sign convention is defined for bents moving or rotating inwards or towards the water. Figure 5-23 shows the soil deformations both right behind and with a distance away from the Bent1 backwall surface, designated as locations A and B in the plot, respectively. The displacements at these two locations are observed showing good correlations with the bridge temperature variations, but perform differently. For example, when the bridge temperature returns back to its original condition after one year of cycling period, as the circle marked in Figure 5-23, meaning almost no temperature changes with respect to that at 08/11/11, the variation of the soil deformation at B returns to zero; while the soil deformation at A keeps changing.

This phenomenon can be more clearly observed in Figure 5-24, where the measured displacement values are divided into two categories taking the time at 02/11/12 as a threshold. In this case, the soil behaviors, before and after that time, will be the responses due to the temperature decrease and increase, respectively. Also, the data of the bridge temperature variations and the corresponding soil deformations are fitted and plotted in Figure 5-24. By comparing the slopes of each linear line, the soil at B behaves almost elastically, showing similar performances during both the temperature increase and decrease stages, while the soil at A shows a larger deformation during the bridge expansion stage after its contraction. Hence, it can be concluded that, due to the integral bridge configuration, the soils behind the abutment will affect the bridge movements. Specifically, for the soil with 2.1m (7ft) away from Bent1, such effects are negligible; for the soil next to the abutment, however, its restraints on the bridge movement will be complicated due to the soil’s plastic behaviors. For example, after the bridge moves away from the soil during the contraction period, the soil behind the abutment may be disturbed or vacated; thus, its restraints on the bridge movements may be decreased when the bridge moves back towards the soil during the next expansion period.
Besides the displacement behaviors, the rotations of the bents are measured at Bent1 and Bent11, shown in Figure 5-25 and Figure 5-26, respectively. Similar to the displacement behaviors, rotations are also negatively correlated with the bridge temperature variations. Again, the rotation at Bent1 shows disturbance and discontinuity at the temperature increase period after the bridge contraction similar to the displacement behaviors, as the circle marked in Figure 5-25.
This scenario can also be attributed to the soil disturbances and its plasticity behaviors. For the rotation of Bent11, as shown in Figure 5-26, since the slab is simply supported on the bent, the soil has negligible effects on it and the induced rotations are approximately 10 times larger than that at Bent1. In addition, compared with the movements of Bent1, the variations of its rotation during the monitoring period, 0.03 degree in this study, is insignificant, and it only contributes about 10% of the Bent1 total displacement. Thus, the Bent1 should be primarily in translations rather than rotations.

**Fig. 5-25** Measured Bridge Temperature and Bent1 Rotation

**Fig. 5-26** Measured Bridge Temperature and Bent11 Rotation
Among all the recorded data, the measured backfill pressures on Bent1 show the most significant discreteness. There seem no clear trends for all the pressure sensor readings, and that may be due to the uneven soil properties, compaction levels, or bridge skew effects. Figure 5-27 shows the representative top and bottom pressure readings at the middle location of Bent1. It can be observed that the pressures have negative correlations with the soil displacements, and the variation of the soil pressures through one year is 20kpa (3psi).

![Figure 5-27 Measured Backfill Pressure at Middle Bent Location](image)

**5.4.5 Pile Strains**

Strains at the four corners of the exterior pile (EP) and interior pile (IP), shown in Figure 5-28, are measured through the pile depths at four elevations from the top to bottom sections, designated as B to E as shown in Figure 5-7, respectively, where the positive x and y axes are defined as perpendicular and parallel to the traffic directions.

![Figure 5-28 Plan View of Pile Diagram](image)

Figure 5-29 shows all the measured strains at four elevations. It can be observed that the strains measured at the top parts of the piles, e.g., B and C sections, are larger than those at the
bottom, e.g., D and E sections. For example, the measured maximum positive and negative strain variations, appearing at the C section, are 40 and 34 microstrain, respectively, even though they are only taking less than 3% of the compressive strength. Also, corresponding to the temperature variations, such as during the decrease period from 06/11 to 12/11, the bridge will contract inward. Under this condition, the outer surfaces of the piles should have been extended with tensile behaviors. This scenario has been justified from the measured strains at sections C to E, where the pile is bending with respect to the negative x-axis; however at B section; the pile is bending with respect to the positive y-axis, and that may be due to the bridge skew effects or the rigid connections between the pile head and bent.

Fig. 5-29 Measured Pile Strains at B to E Cross Section

All the measured strains are further decomposed into four strain components, i.e., the axial strain, x-axis bending, y-axis bending, and torsional strains, according to the following equations from the basic knowledge of material mechanics.

\[
\varepsilon_a = \frac{\varepsilon_1 + \varepsilon_2 + \varepsilon_3 + \varepsilon_4}{4}
\]

\[
\varepsilon_t = \frac{\varepsilon_1 - \varepsilon_2 + \varepsilon_3 - \varepsilon_4}{4}
\]

\[
\varepsilon_x = \frac{\varepsilon_1 + \varepsilon_2 - \varepsilon_3 - \varepsilon_4}{4}
\]

\[
\varepsilon_y = \frac{-\varepsilon_1 + \varepsilon_2 + \varepsilon_3 - \varepsilon_4}{4}
\]
where, $\varepsilon_1$ to $\varepsilon_4$ are the measured strains at the four corners of the pile section shown in Figure 5-28; $\varepsilon_a$ is axial strain; $\varepsilon_x$ is $x$-axis bending strain; $\varepsilon_y$ is $y$-axis bending strain, and $\varepsilon_t$ is torsional strain.

Figure 5-30 and Figure 5-31 show the strains at the top two critical sections of both the EP and IP. It can be observed that, (a) for axial strains, both sections at the two piles show good correlation with the temperature variations. For example, with the increase of temperature, from 12/11 to 08/12, the bridge expands, and the axial strains are also increasing accordingly. For the daily varying trends, however, the two piles are showing opposite signs. This scenario may be due to the different abutment thermal behaviors in the transverse direction, where the decrease of temperature may induce sagging movements at the middle parts of the bents and hogging ones at the two far ends; (b) for the $x$-axis bending strains, they are directly affected by the bridge longitudinal thermal movements. The opposite signs at the B and C sections indicate that a zero moment value, or a double curvature behavior, appears in-between the B and C sections; (c) for the $y$-axis bending strains, they reflect the behavior of the bridge bending transversely with respect to the bridge’s longitudinal $y$-axis. It can be found that the $y$-axis bending strains at the top B section are larger than that of the $x$-axis strains, and that may also be attributed to the bridge’s skew effects or the rigid pile-bent connections. (d) For the torsion strain, the measured values are reasonably as small as expected and can be ignored.

![Fig. 5-30 Strain Components at B-B Section](image-url)
Figure 5-31 Strain Components at C-C Section

Figure 5-32 shows the strains along the depth of the EP at three representative days, i.e., 08/14/11, 11/02/11, and 02/13/12. They are referring to cases with the temperature increasing of 5 °C (10°F), 17 °C (30°F), and 28°C (50°F), respectively, and here the strains at the bottom of the piles are artificially assumed as zero for all cases. Through comparisons, the x-axis bending, induced from the bridge longitudinal deformations, is most sensitive to the temperature variations, even though the y-axis bending cannot be ignored here due to the bridge skew effects. In addition, the double curvature is again observed in the x-axis bending profile, where the zero moment point is approximately located at one third of the total pile length from the pile head.

Fig. 5-32 Measured Exterior Pile Strain at Three Temperature Variations
5.5 Conclusion

Recently, the first two full IABs were built on the soft and stiff soil conditions, respectively, in the state of Louisiana. This paper reports the one-year monitoring results for one of the two bridges, Caminada Bay Bridge on soft soil conditions, from 08/11/11. A total of 81 instrumentations were applied on the bridge to investigate its behaviors in terms of temperature distributions, and the thermal responses of the abutments, slabs, and piles. Some of the important observations are concluded as follows:

(1) The air temperatures measured at the weather station near the bridge site show apparent periodical trends. It is proven that the bridge temperatures are able to be predicted by the air temperatures through curve fitting methods. In addition, based on the measured wind speeds, the convection heat transfer mechanisms are supposed to be significant at the bridge top surfaces, and a highly-varying thermal behavior is observed on the slabs.

(2) The measured slab seasonal temperatures and daily gradients are almost reaching or already exceeding the maximum values that specified by the AASHTO LRFD (2007). In addition, a comprehensive study of the bridge temperature, from the slab top surface to the pile bottom part, indicates that the bridge temperatures are primarily varied within the superstructure and that will ultimately induce the bridge thermal responses.

(3) Both of the strains measured at the bent top surfaces and slab bottom surfaces show a good correlation with the temperature variations. Due to the temperature effects, the tension stresses that appear at the bent top surfaces may possibly crack the concrete, while the compressive stresses are negligible. In addition, there is no difference for strains induced at the locations between bridge ends and middle spans under the current soft soil and stiff structure configurations.

(4) Soils behind the abutment will affect the behaviors of the integral abutment in terms of its displacements and rotations. These effects are complicated, and the soil restraints on the abutment deformations accumulate with time due to the soil’s plastic behaviors. However, for the soil is located away from the abutment, or the connection between the slab-bent is not in an integral type, the soil effects are negligible.
(5) The bending strains of piles with respect to the two main axes, i.e., parallel (y-axis) and perpendicular (x-axis) to the traffic direction, are both important due to the bridge’s skew effects and rigid pile-bent connections at the top parts of the piles, even though the strain values are only taking a small portion of the material compression capacity. For the pile bending profile induced by the slab’s thermal movements, double curvatures are observed, and the zero moment point is located approximately at one third of the pile length from the pile head.

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CHAPTER 6. NUMERICAL STUDY OF AN INTEGRAL ABUTMENT BRIDGE SUPPORTED BY PRESTRESSED PRECAST CONCRETE PILES ON SOFT SOILS - PART B

6.1 Introduction

Integral abutment bridges (IABs) have been designed and constructed since the 1930s in the United States. The original purpose was to replace the traditional jointed bridges, since expansion joints are vulnerable and often affect the sustainability of bridges. Even though with various terminologies, e.g., integral bridge, integral abutment bridge, jointless bridge, rigid frame bridge, U-frame bridge, etc., the IABs are commonly sharing a similar structure configuration. A full IAB, as the one studied here, refers to those bridges whose superstructure (i.e., girder, deck slab, and approach slab) are casted monolithically with the substructure (i.e., abutment and bent) and supported on a single row of flexible piles. Through such integral constructions, not only the leaking and cost issues induced by the expansion joints, as reported by Kong and Cai (2012a), are resolved, many other benefits can also be expected. For example, (1) the bridge design, construction, and replacement, if using IABs, will become much simpler, faster, and require less labor work compared to those using traditional jointed ones; (2) the stresses induced by the traffic loads can be more uniformly distributed along the lateral girders, and the maximum stresses at the continuous bent locations are relatively reduced; and (3) some satisfying performances may be anticipated for IABs against the catastrophic events, e.g., earthquakes, floods, and hurricanes, due to the structural redundancies and the energy absorption mechanisms provided by the soil-structure interaction behaviors (Mistry 2005; Thippeswamy et al. 2002).

Although with such benefits, the IABs have not been widely applied in practice. The reasons are primarily due to the uncertainties on their behaviors under the daily and seasonal temperature variations, creep and shrinkage of materials, etc. No national design specifications exist until now and the current designs and constructions are primarily relying on empirical practice. Some design guidelines have been tentatively developed in several states, while these guidelines are often varied, and sometimes even contradicted between each other. Hence, it is difficult referring the previous experience to the new bridge applications, especially for those with longer spans, greater skews, innovative materials, very soft or stiff soil conditions, etc. In this sense, the numerical modeling methods, using from a simplified 2D to a more complicated 3D finite element model, are often employed in the investigations. For example, Huang et al. (2008)
examined the effects of the structure configurations, i.e., the hinged and fixed connections at the abutment-pile cap, weak and strong axes bending of the steel and concrete piles, etc.; Civjan et al. (2007) discussed the soil effects both from the compacting degrees of the backfills behind the abutment and the soil restraints surrounding the upper part of the piles; and Thippeswamy et al. (2002) compared the responses under the primary and secondary loadings, including the dead load, creep of material, live load, thermal gradient, uniform temperature change, shrinkage, differential settlement, and earth pressure. In addition, some of the other latest numerical studies conducted by some states Department of Transportations (DOTs) and institutes are summarized in Table 6-1. These illustrations are primarily emphasized on the modeling methods of the soil-structural interaction behaviors, and the concerned behaviors on the structural and geotechnical elements.

In the state of Louisiana, the first two IABs were tentatively designed and constructed on the soft and stiff soil conditions, respectively, during the last two years. The corresponding experience will be used to provide references for their future constructions under the soil and environmental conditions in Louisiana. For one of these two bridges, Caminada Bay Bridge on the soft soils, the Louisiana Department of Transportation and Development (LADOTD) and Louisiana State University (LSU) have conducted a field monitoring program on it and the observations over one year from 08/11/11 were separately reported by Kong and Cai (2012a). This paper presents a more in-depth quantitative investigation, based on a numerical modeling method, on the thermal performance of this bridge. Specifically, a 3D finite element model is firstly developed using the commercial software ANSYS 11.0 considering the soil-structure interaction behaviors. Then, the model is verified by comparing the simulated thermal responses with that of the field measurements. Finally, a parametric study is performed to study the behaviors of IABs under different support conditions, temperature loadings, soil types, and structural configurations.
### Table 6-1 Lists of Numerical Studies in Literature

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Model &amp; Tool</th>
<th>Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bemis Road Bridge, Faraji, et al. (1999)</td>
<td>abutment spring: NCHRP (1991); pile spring: API (1993); ( \Delta T = 44.4, ^\circ \text{C} (80, ^\circ \text{F}) ); tool: GTSTRUDL</td>
<td>backfill behind abutment: loose and dense; soil surround piles: loose and dense;</td>
</tr>
<tr>
<td>Lone Tree Road Thippeswamy, et al. (2002)</td>
<td>load: dead load, dead load plus creep, live load, thermal gradient, uniform temperature, shrinkage, differential settlement, earth pressure;</td>
<td>spread foot supports: fixed, hinged; piles orientation: strong and weak axes bending;</td>
</tr>
<tr>
<td>Arockiasamy et al. (2004)</td>
<td>load: displacement due to ( \Delta T = -22.2, ^\circ \text{C} (-40, ^\circ \text{F}) ), shrinkage; tools: FB-Pier, SAP 2000, LPILE;</td>
<td>predrilled hole depth: 2.44 m, 4.88 m; predrilled hole: with and without; pile orientation: weak and strong axes bending; soil in predrilled hole: medium compacted, loose, dense; water table level: 2.44 m, 4.88 m below pile top, and below the pile tip; soil: stiff clay, very stiff clay, dense sand;</td>
</tr>
<tr>
<td>Athens County, Steinberg et al. (2004)</td>
<td>load: ( \Delta T = 28, ^\circ \text{C} (50, ^\circ \text{F}) ); tool: SAP 2000;</td>
<td>skews: 25(^\circ), 35(^\circ), 45(^\circ); span length: 30.5 m, 61 m, 122m; backfill stiffness: 2.71 to 45.1 MN/m(^3);</td>
</tr>
<tr>
<td>Dicleli (2005)</td>
<td>sand-pile: ( p - y ) elastoplastic, horizontal truss elements with plastic axial hinges; abutment-backfill: truss with nonlinear axial hinges; thermal load: longitudinal static pushover load tool: SAP 2000</td>
<td>pile: HP250 ( \times ) 85, HP310 ( \times ) 110; pile orientation: strong and weak axes bending; abutment height: 2 to 5 m; abutment thickness: 1000 and 1500 mm; sand: loose, medium, medium-dense, dense; backfill: compacted, non-compacted;</td>
</tr>
<tr>
<td>Study</td>
<td>Soil-pile spring: multi-linear, COM624P, linear; load: $\Delta T = \pm 44.4^\circ$C (80°F); tool: STAAD Pro</td>
<td></td>
</tr>
<tr>
<td>-------------------------------------------</td>
<td>----------------------------------------------------------------------------------------------------------------------------------</td>
<td></td>
</tr>
<tr>
<td>Fennema et al. (2005)</td>
<td>soil: solid continuum element, strain hardening model in Mohr-Coulomb failure criterion; sand-pile: surface-to-surface contact algorithm; load: (a) 0.023 (m) ($\Delta T = +42^\circ$C) and rotation; (b) 0.013 (m), 0.0013 rad, (c) 0.0096 m, 0.00052 rad; tool: ABAQUS</td>
<td>diameter of the steel sleeve surrounding the piles: 0.6 m, 1 m, 2 m</td>
</tr>
<tr>
<td>Scotch Road Bridge</td>
<td>abutment: nonlinear spring, Modified NCHRP (1991); pile nonlinear spring: API (1993); $\Delta T = +40^\circ$C (72°F) and $-55^\circ$C(99°F); tool: GTSTRUDL</td>
<td></td>
</tr>
<tr>
<td>Khodair and Hassiotis (2005)</td>
<td>connection: fixed, hinged; pile: HP, CIP, Pile orientation: strong and weak axes bend; girder depth: 0.91 m, 1.14 m, 2.08 m; length: 66 m, 132 m; wingwall: 45°, parallel, perpendicular; length: 2.5m, 4.3m</td>
<td></td>
</tr>
<tr>
<td>Orange Wendell Civjan et al. (2007)</td>
<td>soil: loose sand, soft clay, stiff clay, very stiff clay</td>
<td></td>
</tr>
<tr>
<td>Bridge 55555</td>
<td>soil-pile: $p - y, f - z$, $q - z$, COM624P; backfill-abutment: $F - \Delta$; load: $\Delta T = \pm 28^\circ$C (50°F); tool: ANSYS;</td>
<td></td>
</tr>
<tr>
<td>Huang et al. (2008)</td>
<td>connection: fixed, hinged; pile: HP, CIP, Pile orientation: strong and weak axes bend; girder depth: 0.91 m, 1.14 m, 2.08 m; length: 66 m, 132 m; wingwall: 45°, parallel, perpendicular; length: 2.5m, 4.3m</td>
<td></td>
</tr>
</tbody>
</table>
**Table 6-1 Lists of Numerical Studies in Literature (Continued)**

<table>
<thead>
<tr>
<th>Study Description</th>
<th>Parameters and Details</th>
</tr>
</thead>
</table>
| FB & SB Bridge             | Dicleli and Erhan (2008)  
backfill: linear springs, subgrade reaction modulus $k_{sh} = \frac{14500}{H} \times z$;  
soil-pile: $p - y$ linear springs with elasto-plastic curve;  
load: AASHTO HL-93  
Tool: SAP 2000  
span: 19.8m, 39.6m;  
girder: W760 $\times$ 173, AASHTO VI;  
pile: HP250 $\times$ 85, HP310 $\times$ 125;  
orientation: strong, weak bending;  
abutment height: 3 m, 5 m;  
abutment thickness: 1 m, 1.5m;  
wingwall: with and without;  
backfill: with and without;  
backfill compaction level: 18 kN/m$^3$, 20 kN/m$^3$, 22 kN/m$^3$;  
soil: soft, medium, medium stiff, stiff clay |
| No. 203, 211, 222          | Pugasap, et al. (2009)  
abutment nonlinear spring: classic earth pressure;  
pile nonlinear spring;  
both soil considers cyclic behavior;  
load: thermal, time-dependent as equivalent temperature  
tool: ANSYS |
| Bridge Kii                 | Ooi et al. (2010)  
bilinear elasto-plastic and linear elastic Mohr-Coulomb  
tool: PLAXIS & FOUNATION |

### 6.2 Project Description

The detailed bridge configurations can be referred to Kong and Cai (2012a), and some of the important information are briefed here for the convenience of readers. The bridge is located at Grand Isle, LA ($29°15'48"$ N $89°57'24"$ W), about 160km (100 miles) to the south of New Orleans, LA. The total length of the bridge is 1202m (3945ft), while the modeled part is for the first 11 spans, shown in Figures 6-1 and 6-2, including a 3m (10ft) sleeper slab, a 12m (40ft) approach slab, a 91m (300ft) continuous concrete slab, and the substructure underneath, such as
the abutment, pile, and soil. Each bent is supported by a single row of four precast prestressed concrete (PPC) piles with a diameter of 1m (36in). In addition, the material properties designed for this bridge are summarized as follows: (a) Class AA (M) concrete, with a strength of 28Mpa (4000psi), is used for the slabs and bents; (b) Class P (M) high performance concrete, with a minimum compressive strength of 41Mpa (6000psi) at 28days, and an average compressive strength of 69Mpa (10000psi) at 56 days, is for the PPC piles; (c) Type 316LN stainless steel, with an elastic modulus of 200Gpa (30000ksi), a tensile strength of 515Mpa (75ksi), and a yield strength of 205Mpa (30ksi), is for the deformed reinforcing steels in the bents and slabs; (d) Grade 60 black steel, with a 414Mpa (60ksi) yield strength, is for all the other deformed reinforcing steels; (e) Grade 270 steel, with a 1860Mpa (270ksi) yield strength, is for the prestressing strands; and (f) the thermal expansion coefficient of the concrete is assumed as 5 × 10⁻⁶/°F after a synthesized consideration of the specified values 6 × 10⁻⁶/°F from AASHTO LRFD (2007), and the lower and upper bounds of 4.7 × 10⁻⁶/°F and 6.5 × 10⁻⁶/°F from ACI 209 R-92 (2004).

Fig. 6-1 Elevation View of the First 11 Span of the Caminada Bay Bridge

Fig. 6-2 Plan View of the First 11 Span of the Caminada Bay Bridge
6.3 Model Development

6.3.1 Boundary Condition

Different from some other commonly designed IABs, where integral joints are primarily constructed at the interfacial location between the slab-girder-abutment of the bridge’s two ends, the bridge discussed in this study, otherwise, has more rigid connecting behaviors throughout the whole bridge length. First, at the left end of the bridge, labeled as A in Figure 6-1, a 10.2cm (4in) expansion joint was provided between the sleeper slab and the approach slab; and the approach slab, in turn, is laid on a reinforced rubber pad. Thus, certain friction restraints, if not many, may exist and obstruct the free movements of the approach slab. Second, for all the interior bents from Bent1 to Bent10, shown as the representative label B in Figure 6-2, tensile steel rebars are constructed both extending from the bents to the slabs, and also from the pile heads to the bents. Thus, continuous behaviors are expected at these locations. Third, at Bent11, a strip seal joint is provided between the slab and Bent11 so that the longitudinal movement is not fully restrained and the rotation is released. Therefore, the boundary conditions for this bridge are assumed and modeled as simple support conditions at the two ends (points A and C) and fixed ones (point B) in between.

6.3.2 Soil-Structural Interaction

Two boring logs, at the location shown in Figure 6-2, are available from the approach slab to Span10. The soil information obtained at the station 99+74, shown in Figure 6-3(a), was adopted to represent the soil condition of the modeled bridge. After combing the similar properties together, the soil layers that will be used in the numerical model are shown in Figure 6-3(b), where it can be roughly categorized as a layer of medium clay for the backfill, and followed by two layers of medium sand and medium clay below the water table and surrounding the piles.

Generally speaking, in modeling of the soil-pile interaction behaviors, the p-y curve and elastic continuum methods are commonly proposed in the literature. Specifically, the former one based on the Winkler hypothesis is simple but have been widely applied in the routine design, where the soil is simplified as a set of discrete elements that the soil response at one point is independent on the pile deflection elsewhere. In addition, the soil-structure interaction behavior is accounted by a series of p-y curves along the pile depths, where the p and y refer to the soil
force and pile deflection, respectively. The generations of these p-y curves are complicated and could be affected by many parameters, e.g., soil type, shear strength, moisture condition, effective stress, stress history, loading condition, etc. However, these curves, nowadays, can be more conveniently obtained from some commercial or free software and programs, e.g., COM624P (1991), LPILE, FB-Pier, etc. In this study, the COM624P (1991) program is employed, in which the coded p-y curves in this program are based on the full-scale experiments; thus the continuum effects are explicitly implemented.

![Fig. 6-3 (a) Soil Layout from Boring Log 1; (b) Soil Layers for FEM Analysis](image)

In modeling of the backfill-abutment interaction behaviors, several other design curves are available for sand soils, e.g., NCHRP (1991), CGS (1992), and Duncan and Mokwa (2001). Among all the curves, the NCHRP (1991) curve, shown in Figure 6-4, is commonly adopted in the design. Specifically, the force and displacement relationships between the backfill and abutment can be expressed as,
where \( F = \) effective soil lateral resistance, \( K \) = coefficient of lateral earth pressure for the passive \( K_p \) and active \( K_a \) conditions, respectively, determined by the ratio of the wall deformation and height \( (D/H) \), \( \sigma'_v \) = vertical effective soil stress, equal to the soil density multiplied by the depth of the soil \( (\gamma'z) \), \( wh \) = width and height dimensions of the tributary area of the abutment backwall. Figures 6-5(a) and 6-5(b) show the two representative force-displacement \( (F-D) \) curves for the loose and dense sand conditions along the three depths behind Bent1, respectively, and they will be used in the following parametric study section. For the cohesive backfill, however, there is no design curve available based on the author’s knowledge. According to CALTRANS (2004), creep effects should be considered in estimating the design earth pressures for cohesive soils, and they are complicated and require laboratory tests. Thus, the cohesive or other fine-grained soils are often avoided for backfill materials.

![Diagram](image)

**Fig. 6-4** Relationship between Wall Movement and Earth Pressure from NCHRP (1991)

![Diagram](image)

**Fig. 6-5** F-D between Bent1 and Backfill at Three Elevations (a) Loose Sand; (b) Dense Sand
6.3.3 Numerical Model

Based on the above discussions, a 3D finite element model is developed using the commercial software ANSYS 11.0 as shown in Figure 6-6. Specifically, (1) the 3D Solid45 element, with eight nodes and three degrees of freedom for each node, is adopted for the slabs and bents; and the uniaxial Beam4 element, with six degrees of freedom for each node, is for the piles; (2) the unidirectional COMBIN39 spring elements, with the nonlinear force-deflection capability, are adopted for the soil-pile interaction behaviors both in the parallel and perpendicular directions with respect to the bridge traffics; (3) for the backfill-abutment interaction behaviors, they are ignored at this stage considering the complexity of obtaining the cohesive backfill behaviors. This assumption, on one hand, can be justified from the field measurements, referring to Kong and Cai (2012a), where the variations of the pressures for such soil types under current IAB configurations are observed negligible; on the other hand, the insignificant effects of the backfills on this bridge are also proven later in the parametric study section; and (4) for the connection behaviors between the piles and bents, a multipoint constraint MPC184 element is used and the rigid beam connecting option is selected.

Fig. 6-6 3D FEM of Caminada Bay Bridge Using ANSYS
Besides the structure and soil models, the load model is another important aspect. Uniform and gradient variations are two major temperature components in the bridge thermal analysis, where the former one will affect the bridge deformation; and the later one, being further categorized as the linear or nonlinear distributions, will induce the self-equilibrating thermal stresses. As reported by Kong and Cai (2012a), the major temperature variations of this bridge are primarily appearing at the superstructure based on the field measurements. Since only the surface temperatures of the slabs are measured, then, the temperature distribution patterns through the slab depth are predicted here. Using the available temperature predication methods (Elbadry and Ghali 1983), the bridge temperatures during the two representative hottest and coldest weeks, i.e., Jan 1st to Jan 13th and Sep 6th to Sep 16th, are selected and simulated. The boundary conditions, i.e., ambient temperature and wind speeds, are referred to the local weather station (Grand Isle, LA 29.263 N, 89.957 W http://tidesandcurrents.noaa.gov), and the solar radiation is calculated through an algorithm by considering the relative position relationships between the solar and bridge.

Figure 6-7 shows the comparisons of the temperatures, at the bent top and slab bottom surfaces, between the field measurements and ANSYS predictions for the two selected weeks, where the observed good fitting trends verify the rationale of the temperature prediction model. In addition, the largest temperature gradients during the two modeling ranges are plotted in Figure 6-8(a), and the corresponding normalized results, by subtracting the initial values, are shown in Figure 6-8(b). At the same time, the corresponding temperature design values for such concrete slabs in the current bridge site, referred to AASHTO LRFD (2007), are also superposed in the plot. It can be observed that the temperature differences between the top and bottom slab surfaces are as high as 11°C (20°F), but still within the code specifications. In addition, the gradient distribution through the depth of the slab is nonlinear; even though the nonlinear magnitude is not significant and could be more conveniently represented by a simplified linear one in design when the temperatures are only measured and available at the slab surfaces as that in the current case.
6.3.4 Numerical Study Results

The bridge thermal behaviors, i.e., abutment displacements and rotations, slab top and bottom surface strains, and pile bending strains with respect to the x and y axes, marked in Figures 6-9(a) to 6-9(c), during a 24-hour at the two representative hot and cold days, 08/30/11 and 2/12/12, respectively, are simulated. The field measured temperatures at the top and bottom
surfaces of the slabs, shown in Figures 6-10(a) and 6-10(b), are directly applied as node loads, and a linear temperature distribution is assumed through the slab depth.

(a) Gages Mounted on Bent Top and Bottom Surfaces (Plan and Elevation View)

(b) Instrumentations Mounted on Bent1 (side view)

(c) Instrumentations Mounted on Bent1 (Plan View)

Fig. 6-9 Instrumentations Applied on Integral Abutment Bridge
The comparisons between the simulations and field measurements are shown in Figures 6-11 to 6-16, where all the values are normalized by subtracting the results at the initial time. Generally, the results show similar trends between the simulations and measurements for all the comparing items, so that the proposed numerical model can reasonably represent the bridge and environmental conditions. Specifically, first, the obvious, but not significant, discrepancies are appeared in the displacement and rotation plots, and they may be attributed to (a) the uncertainty on the soil types and the corresponding compaction degrees behind Bent1; (b) the modeling error of the support and boundary conditions at the bridge ends; (c) the modeling error of the soil-pile interaction behaviors at Bent1; and (d) the long term cumulative effects due to the soil plasticity behaviors behind Bent1. Second, for the strains in the slabs and piles, however, the modeled results match well with the measurements at the selected time. In addition, it should be noted that, during the model calibration process, this bridge was found extremely sensitive to the loading types (uniform or gradient temperatures), support conditions (free or fixed), flexure rigidities (young’s modulus of piles, soil types surrounding the piles, and pile-bent connections), but quite insensitive to the backfill properties. Also, the behaviors at the middle parts of the bridge seem similar to that at the left integral end (Bent1 location). Then, these features of IABs are studied in the next parametric study section.
Fig. 6-11 Comparisons of Bent1 Displacements on (a) 08/30/11 and (b) 02/12/12

Fig. 6-12 Comparisons of Bent1 Rotations on (a) 08/30/11 and (b) 02/12/12

Fig. 6-13 Comparisons of Bent Top Surface Strains on (a) 08/30/11 and (b) 02/12/12
Fig. 6-14 Comparisons of Slab Bottom Surface Strains on (a) 08/30/11 and (b) 02/12/12

Fig. 6-15 Comparisons of Pile X-axis Bending Strains on (a) 08/30/11 and (b) 02/12/12

Fig. 6-16 Comparisons of Pile Y-axis Bending Strain on (a) 08/30/11 and (b) 02/12/12
6.4 Parametric Study

A parametric study is conducted to investigate the behaviors of such IABs if being applied to other soil conditions and structure configurations. Generally, many factors may affect the performance of a bridge. However, the restraints to the thermal movements are of significant importance for IABs. These restraints may come from the soil resistances, structural rigidities, support conditions, element connecting behaviors, etc. For example, if the restraints are weak, a larger displacement from the superstructure may cause the pile buckling or soil failures; otherwise, if the restraints are strong, the induced internal forces may be substantial and that may damage the superstructure elements. For these reasons, the following parameters are varied in the investigations, including (a) support conditions, i.e., simply and fixed supports at the two ends of the bridge; (b) soil types behind the abutment and surrounding the piles, i.e., loose sand, dense sand, and stiff clay; and (c) joint connections at the interfacial locations between the pile and bent, i.e., fully rigid and roller supported.

In addition, since the primary concerns are focused on the bridge thermal behaviors, then, both the uniform and gradient temperature variations, referred to the temperature design criteria in the AASHTO LRFD (2007), are considered. For the uniform temperature variations, based on the material type and bridge location, a 44.4°C (80°F) temperature increase is assumed and applied to the concrete slab. For the thermal gradients, considering the situation that the bridge is located at Zone two (one of the four solar radiation zones where the positive and negative temperature distribution patterns being accordingly stipulated), and also the material type, superstructure depth, and wearing surface, the top and bottom surfaces of the concrete slab are assigned with temperatures of 46°F and 0°F, respectively; and a linear gradient distribution is assumed through the slab depth. Therefore, by varying all these parameters, listed in Table 6-2, the performances of the bridge are discussed in terms of the displacement of the abutment and pile, the backfill pressures behind the abutment, and the forces in the slab and piles. For the convenience of discussions, the legend for each parametrical studying case is uniformly defined here by its loading conditions and the varying parameters, i.e., U and G refer to the uniform and gradient temperature loadings, respectively. For example, the legend of the G_Fixed Support means the case of the bridge being subject to gradient temperatures and rigidly supported.
Table 6-2 Parametric Study Cases

<table>
<thead>
<tr>
<th>Cases</th>
<th>Effects</th>
<th>Temperatures</th>
<th>Backfills</th>
<th>Soils</th>
<th>Connections</th>
<th>Supports</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case1</td>
<td>Support</td>
<td>U_ 80°F</td>
<td>Loose Sand</td>
<td>Soft clay</td>
<td>Rigid</td>
<td>Free</td>
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<tr>
<td></td>
<td></td>
<td>G_ 46°F</td>
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<td></td>
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<td>Fixed</td>
</tr>
<tr>
<td>Case2</td>
<td>Backfill</td>
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<tr>
<td></td>
<td></td>
<td>G_ 46°F</td>
<td>Dense Sand</td>
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<tr>
<td>Case3</td>
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<td>G_ 46°F</td>
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<td></td>
<td>G_ 46°F</td>
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<td></td>
<td>Roller</td>
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</tr>
</tbody>
</table>

6.4.1 Effects of Support Conditions

The bridge responses under different support conditions are shown from Figures 6-17(a) to 6-17(d). Generally, in the numerical modeling or routine design, the supports are most of the time idealized as two extreme conditions with either fully free or fixed; in reality, however, these assumptions may hardly happen. There must be some possibilities that either the free deformations are blocked to a certain degree, or the current manufacture or construction technic cannot developed a perfectly rigid connection. For IABs, the effects of supports on the behaviors of the bridges are supposed to be more obvious than that on traditional jointed ones.

In these figures, it is reasonable to observe that, the case with free support and uniform temperature variations provides the most significant effects on the bridge performance. For example, it causes comparatively larger displacements, and that in turn, induce the correspondingly higher backfill pressures on the abutment and greater positive and negative bending moments along the pile. On the other hand, the effects of the support conditions on the slabs are much more complicated. For the uniform temperature increase condition, the significantly higher compressive strains are generated when the bridge movements are fixed; while for the gradient distribution condition, the positive strains are appearing at the slab surfaces under the free supports condition. In addition, the slab strains distribution along the bridge length show no differences at the locations between the bridge ends and middle parts. It may indicate that the only differential element between these two locations, i.e., backfills behind Bent1, shows negligible effects on the bridge compared to other structural or geotechnical elements, such as the soils surrounding the piles, pile rigidities, or connections between the pile and bent. Based on this observation, then, it is not advisable to allow a large movement at the
superstructure for the sake of the substructure’s safety; however, restraining the thermal movements is also not beneficial for the behaviors of superstructure.

![Graphs and Figures](image1.png)

**Fig. 6-17 Bridge Responses under Different Support Condition Cases**

### 6.4.2 Effects of Backfill behind Abutment

The bridge responses under different backfill material types are shown in Figures 6-18(a) and 6-18(b), where only the behaviors of displacements and backfill pressures are plotted since the variation of the responses at the slab and pile are negligible. As is shown in the figures, there are almost no differences in terms of the displacements between the dense and loose sand backfills. This phenomenon again verifies some of the previous arguments that, with the current structure configurations, the bridge rigidities should be largely contributed by the big cross-
section PPC piles and rigid pile-bent connections, whereas the shallower depth of the backfill nearly provides no resistances. However, comparing the performances between the two backfill pressures due to the large superstructure movements, the dense sand already reaches its passive critical conditions with the pressures from the top to the bottom as 27kpa (4psi), 82kpa (12psi), and 138kpa (20psi), respectively; while the loose sand pressures through the depth, with 5kpa (0.78psi), 16kpa (2.3psi), and 25kpa (3.7psi), respectively, are still within its passive critical values of 12kpa (1.7psi), 36kpa (5.2psi), and 59kpa (8.6psi), respectively.

![Fig. 6-18 Bridge Responses under Different Backfills behind the Bent1 Cases](image)

6.4.3 Effects of Soil Surrounding Pile

Figures 6-19(a) to 6-19(d) show the bridge responses under the conditions with different soil types surrounding the piles. In these figures, apparently different behaviors can be observed at the substructure. For example, under the soft clay case, the induced displacement at the slab top surface is almost 1.5 times larger than that under the dense sand condition. Similar to the previous discussion on the support effects, the weak restrains from the soft soils surrounding the piles will allow a larger pile deformation and that induces greater soil pressures behind the abutment and higher bending forces in the piles. As for the soil effects on the superstructure, the induced strains in the slabs will increase about 48% if changing the soft clay to dense sand under uniform temperature variations. Under gradient temperature distributions, however, it does not show much difference for either soil types. It is again observed that, under free support boundary conditions, the gradient temperature loading is observed to be the critical case which provides
much higher positive and negative strains in the slabs than that from the uniform temperature variations.

Fig. 6-19 Bridge Responses under Different Soils Surrounded the Piles Cases

6.4.4 Effects of Pile-Bent Connection

Figures 6-20(a) to 6-20(d) show the performances of the bridge with different pile-bent connections. As can be observed, the significant differences, by changing roller connections to rigid ones, are lying in the induced forces on the piles. Specifically, the rigid connections will induce larger forces at the pile-bent interfacial location; while the forces are zero for roller cases at the connecting locations, and the maximum values appear approximately at the one third parts below the pile head. Besides that, different abutment rotation behaviors, even though not apparent, are shown in the detailed plots of Figure 6-20(a). The rigid connection provides
continuous but larger rotations compared to the discontinuous and smaller ones under the roller connections. Those differences, to some degrees, cause relatively larger backfill pressures at the roller connections than at the rigid ones. The soil effects on the superstructure, however, are not substantial. Even though the strains under the roller case can be about 40% smaller than that under the rigid connection for uniform temperatures variations, yet the dominant or critical temperature loadings are still from the gradient temperature which shows only 5% differences between the two soil types.

![Bridge Responses under Different Connection Details between Slabs and Bents Cases](image)

**Fig. 6-20** Bridge Responses under Different Connection Details between Slabs and Bents Cases

### 6.5 Conclusion

During the recent several decades, integral abutment bridges, with various advantages and benefits over the traditional jointed ones, have been designed and constructed. However, many
uncertainties of such bridges still exist, such as the bridge thermal behaviors and the complicated soil-structural interaction behaviors. Recently, the LADOTD has built its first full IAB, Caminada Bay Bridge, on the soft soil condition. Based on the configurations of this bridge, together with the one year’s monitoring results, a 3D numerical model method is proposed and validated, where the pile-soil and abutment-backfill interaction behaviors are incorporated. With the verified model, the concerning structural and geotechnical parameters are varied through a parametric study to investigate their effects on the bridge thermal performances. Some of the conclusions are drawn as follows:

(1) The support conditions in the reality situations may not be completely the same as that were designed for, but they are crucial for bridges without joints. For IABs, it is not advisable to allow a fully free supports at the superstructure, since the induced movements will cause larger backfill pressures and pile internal forces; whereas, it is also not a good idea to adopt fully fixed supports since they will induce larger slab internal forces. This dilemma may be the direct reason that limits the applications of integral constructions on much longer span bridges, since the larger thermal movements, together with their corresponding thermal effects, have to be either accommodated by the substructure or the superstructure.

(2) For this specific bridge, the effects from the backfill are negligible. The height of the abutments are shallower compared with the much deeper and bigger cross section piles. Thus, the bridge deformations are actually controlled by the pile rigidities, soil resistances surrounding the piles, and connection behaviors between the pile-bent. However, if for some other more general IABs, the stiff soils may induce higher internal forces at superstructure; and the soft soils provide greater forces at substructure.

(3) Soils surrounding the piles show the most obvious effects on the bridges. If changing the soft soils to the stiff ones, it will provide a maximum of 1.5 times smaller bridge displacements and 20% smaller backfill pressures; but at the same time, it will also induce 70% larger pile positive strains and 48% larger slab negative strains. Thus, it seems difficult to design an optimized structure that can simultaneously benefit both the superstructure and substructure.

(4) The connection behaviors between the pile-bent affect the bridge responses in two aspects. On one hand, they switch the locations of the maximum pile internal forces from the
rigid connection case, i.e., at the interfaces between pile-bent, to the roller connection case, i.e., at the top third parts below the pile heads. On the other hand, they also affect the rotation behaviors of the abutment, and that in turn, to some degree, affects the backfill pressures and slab strains even though these changes are not significant due to the small sizes of abutment in this bridge.

In summary, for IABs, the thermal movements have to be accommodated by either the superstructure or substructure: (1) if the superstructure is allowed to move comparatively free, then, the substructure will experience large deformations and internal forces; (2) if the superstructure is not allowed with a larger movement, by adopting fixed boundaries, more stiffer backfills, or other approaches, then, greater internal forces may be induced on the superstructure and that will be beneficial for the substructure. These dilemmas will become more complicated if considering the combinations of other different loading conditions, such as the temperature uniform variations, gradient distributions, dead loads, and live loads. In this sense, some of the settling methods for this argument may lie in the attempts of new material types with different physical and mechanical properties. The efforts to that aspect can be referred to Kong and Cai (2012b) where glass fiber reinforced polymer panel (GFRP) slabs are proposed, and the thermal behaviors of IABs with GFRP slabs are tentatively studied.

6.6 References


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Caltrans (2004). Bridge design specifications, California Dept. of Transportation, CA.


CHAPTER 7. FRP PANEL THERMAL PROPERTY HOMOGENIZATIONS AND APPLICATIONS TO INTEGRAL ABUTMENT BRIDGES

7.1 Introduction

Integral abutment bridges (IABs) have been designed and constructed during the past several decades. One of the most special features of such bridges is that the expansion joints are eliminated at the abutments and/or along the length of bridges. Then, when they are subjected to the temperatures or other loadings, both the superstructure (i.e., the approach slab, deck slab, girder, etc.) and substructure (i.e., the abutment, pile, backfill behind the abutments, and soil surrounding the piles, etc.) will work as a whole unit to provide resistances. Many benefits can be expected from IABs, e.g., more efficient design and construction, more uniform live load distribution, better performance under catastrophic events, etc. However, some arguments are still unsettled that the benefits of IABs have not been adapted to different kinds of bridges, especially for those with longer spans, higher skews, extreme soft or stiff soil conditions, and innovative structural configurations and material types. Therefore, (1) the current designs and constructions of IABs are primarily relying on empirical experience and there exist no national design codes; (2) the effects of the soils, either behind the abutments or surrounding the piles, on the performances of the structure elements require further investigations; and (3) the maximum design criteria, from the aspects of the bridge geometries and configurations to the modeling and designing methods, still varies from states to states (Mistry 2005; Thippeswamy et al. 2002; Dicleli 2005; Fennema et al. 2005; Khodair and Hassiotis 2005; Civjan et al. 2007; Huang et al. 2008; Dicleli and Erhan 2008; Pugasap et al. 2009; Ooi et al. 2010; Kong and Cai 2012a).

Among all the studies, Kong and Cai (2012b) have recently reported the concerns on the behaviors of the IABs’ slabs under the seasonal and daily temperature variations. As was indicated, there is a dilemma during the designs that it is difficult to both release the restraints to the thermal movements from the superstructure and, at the same time, reduce the forces generated on the substructure. For example, if the restraints to the thermal movements are too strong, the induced larger thermal forces may damage the slabs and girders; otherwise, if the restraints are too weak and the free movements are allowed, the larger bridge displacements may produce greater forces on the substructure and cause failures on both of the piles and soils. Essentially, all these responses, to a certain degree, are determined by the material properties of
the slabs and girders. For concrete superstructures, the comparatively larger thermal expansion coefficients but smaller tensile strength capacities limit the maximum thermal movements that can be accommodated, and which, in turn, restrains the maximum bridge lengths of IABs that can be designed.

Under this circumstance, one of the alternative attempts that may resolve the above issues is to change the concrete materials to a new and innovative type, such as the fiber reinforced polymer (FRP) composite materials discussed in this study. FRP materials have already been widely applied in the military industry and aerospace field for a long time. In bridge engineering, however, they just begin to be adopted in practice in the recent decades. Among all the utilizations, the glass FRP (GFRP) panel, with light weight, high strength, good corrosion resistance, and long term durability, is considered to be one of the most prosperous alternatives and have already been applied to the bridge replacements, retrofits, and rehabilitations.

The behaviors of GFRP panels in bridge engineering have been widely investigated in the recent decades, such as in the aspects of the modeling and designing methods (Cai et al. 2009; Davalos et al 2001), static performances (Camata and Shing 2005; Zhang and Cai 2007; Turner et al. 2004); and dynamic performances (Zhang et al. 2006; Aluri et al. 2005). Also, other tentative researches are performed on their thermal behaviors in terms of the temperature distributions, thermal deformations, strains, and stresses. The thermal properties of the GFRP panels, i.e., the solar absorption ability, convection coefficient, conduction coefficient, and expansion coefficient, are all different compared to that of the concrete and steel. Thus, the induced thermal responses are expected to be distinct (Laosiriphong et al. 2006; Liu et al. 2008; Reising et al 2004).

### 7.2 Motivation and Scope

In this paper, based on the study of one as-built IAB, the Caminada Bay Bridge designed by the Louisiana Department of Transportation and Development (LADOTD), a tentative effort is made to investigate the thermal behaviors of such an IAB after replacing their concrete slabs with FRP panels. To the authors’ knowledge, using FRP panels on IABs is all new, except for the Market Street and Laurel Lick bridges proposed by the Constructed Facilities Center at West Virginia University (Shekar et al. 2005). In this sense, the configuration of the bridge in this study is artificially assumed by applying one popular FRP panel type that is available in the
market. FRP panels are often categorized based on three manufacture methods as (1) pultrusion, e.g., Martin Marietta Composite, (2) vacuum-assisted-resin-transfer-molding (VARTM), e.g., Hardcore Composites, and (3) open mold hand lay-up, e.g., Kansas Structural Composite (Morcous et al. 2010).

In this study, the GFRP panels provided by KSCI are adopted since they have already been applied in practice through some projects and gained much success. More importantly, a large number of analytical or experimental studies are conducted on such panels; thus the corresponding material properties, both from the micro and macro aspects, are available in the literature and can be adopted in the following numerical modeling studies. However, it should be noted that, (1) the results in this study are specific for, but not limited to, the behaviors of IABs with GFRP panels. They can also provide some meaningful overviews for the performances of other general IABs when adopting different FRP materials and configurations; (2) this study only investigates the thermal behaviors of the bridges. Then, the responses due to the traffic, self-weight, or other loadings are out of the scope of this research, even though the method introduced here are still suited for further investigations when considering other loadings; and (3) the applications of the GFRP panels do not necessarily mean that they will provide the best thermal performance and, at the same time, meet all the other structure and construction requirements. After all, the original designs of such FRP panels are for the bridges’ rehabilitations and retrofits, and that are primarily due to their higher strength to density ratio and better corrosion resistance capacities. However, one of the most prosperous benefits of FRP composite materials is that, their properties can be designed to meet certain requirements by adjusting the fiber and matrix types, constituent proportions, and the orientations of the lamina and laminate. In this sense, the conclusions obtained here may also provide meaningful references if designing some specific FRP panels for IABs in the future. Therefore, first, a homogenization and stiffness-equivalent method is proposed and employed to predict the equivalent elastic and thermal properties of the GFRP panel. Then, the behaviors of the Caminada Bay IAB, before and after replacing concrete slabs by GFRP panels, under the uniform and gradient temperatures specified by AASHTO LRFD (2007) are discussed.
7.3 Numerical Model Development

7.3.1 Model of IAB

The studied Caminada Bay IAB is located at Grand Isle, LA, about 160km to the south of New Orleans, LA. The detailed information in terms of the bridge configuration and soil information can be referred to Kong and Cai (2012a, 2012b), where both the field monitoring and numerical investigations on this bridge have been separately reported. Some of the important information is illustrated here for the convenience of readers. The total length of the bridge is 1202m, while the integral part is at its first 11 spans, shown in Figure 7-1, including a 3m sleeper pad, a 12m approach slab, a 91m continuous concrete slab, and also the concrete bents, prestressed precast concrete (PPC) piles, and soils underneath. The slabs are fully integrated with Bent1 at the left end, rigidly connected with all interior bents from Bent2 to Bent10, and simply supported on Bent11 at the right end. The bridge model, shown in Figure 7-2, is established using the commercial software ANSYS 11.0, where the soil-structural interaction behaviors between the pile-soil and abutment-backfill are considered using the p-y curve method.

Fig. 7-1 Elevation View of the First 11 Spans of Caminada Bay Bridge

Fig. 7-2 3D FEM of Caminada Bay Bridge Using ANSYS
7.3.2 Prediction of GFRP Panel Properties

The adopted GFRP sandwich panel, shown in Figure 7-3, is made of E-glass fibers and polyester resins consisting of two facial laminates and one core. The detailed information in terms of the fabrication techniques, geometry descriptions, and constituent layouts are referred to Cai et al. (2009), Oghumu, S. O. (2005), and Qiao and Wang (2005).

![Figure 7-3 GFRP Hollow Section Sandwich Panel](image)

Compared to other commonly utilized homogeneous and isotropic concrete slabs, one of the difficulties in studying these heterogeneous and non-isotropic FRP panels is to obtain their material properties and to develop the efficient numerical models. For this specific GFRP panel, the properties of the lamina and laminate are simplified and approximately predicted using the micro and macro mechanics method. However, developing a full finite element model of such panel, if according to its original sinusoidal and hollowed configurations, will generate a huge amount of nodes and elements where the computation will be extremely time-consuming. Several simplifying modeling methods are proposed including the one-layer model, three-layer model, simplified I-beam model, etc. (Cai et al. 2009; Davalos et al. 2001; Morcous et al. 2010). Among all the approaches, the homogenized one-layer model is efficient and appropriate in this study. This method has been successfully adopted in the study of the bridges’ global static and dynamic behaviors, e.g., the live load distribution factor and dynamic allowance factor; however, it has not been used in the thermal studies. The basic principle of this approach is to simplify the whole complicated GFRP panel to a homogeneous one-layer structure having the equivalent properties, i.e., the axial stiffness, bending stiffness, and shear stiffness. Besides the global displacements obtained from the equivalent slab model, the stresses of the original GFRP panel can be further acquired by: (1) outputting the internal forces from the equivalent slab at the
sections that are interested, i.e., axial force (N), bending moment (M), etc., (2) applying these forces onto the original GFRP panel with its own geometry characteristics of the cross section, i.e., area (A), moment of inertia (I), etc., and (3) using the basic knowledge of the material mechanics and beam theory, expressed by Eq.1 and Eq. 2, to separately calculate each stresses components and superpose them together afterwards.

\[
\sigma_{\text{axil}} = \frac{N}{A} \quad (1)
\]

\[
\sigma_{\text{bend}} = \frac{My}{I} \quad (2)
\]

where \(\sigma_{\text{axil}}\) and \(\sigma_{\text{bend}}\) are the axial and bending stresses; N and M are the axial force and bending moment on the section; A and I are the section area and moment of inertia; y is the distance between the location where the stress to be calculated to the neutral axis of the section.

This study firstly attempts to homogenize the thermal properties of the complicated GFRP panels by using an equivalent solid slab. Since the original GFRP panel is in hallowed configuration, the solid equivalent slab may not simultaneously have an identical young’s modulus (E) for the axial stiffness (EA) and bending stiffness (EI) of the sections, respectively. However, any linearly distributed temperatures can be decomposed into two components, including the uniform temperature variations that will cause axial movements and forces, and the linear gradient ones that will induce pure bending deformations and moments. Hence, the stiffness EA and EI of the equivalent slab are developed firstly; then, when calculating the thermal responses of the slabs, the two temperature loadings are separately applied on the bridges corresponding to the pure axial or bending conditions, and the total responses of the slabs will be the superposition of the two results.

Following this concept, a representative GFRP slab with the size of 0.91m×1.52m×0.13m (36in×60in×5in), together with the same size equivalent (EQUIV) slab, are shown in Figure 7-4, where both the properties in the two main X and Y directions are developed for this orthotropic GFRP slab. The procedures of the properties development in the X-direction, similar to that in the Y-direction, are illustrated as follows: (a) applying the same axial forces at the end of two GFRP and EQUIV slabs under simply supported conditions, to get the equivalent axial \(E_{\text{axial,x}}\)
by making the two slabs have the same axial deformations at the middle span; (b) applying a uniform temperature increase of 13°C (23°F) on two slabs under fixed support conditions where restraining the degrees of freedoms (DOFs) in the X-direction, to get the equivalent thermal coefficient $\alpha_{\text{axial, x}}$ in the X-direction by making the two slabs have the same axial forces; (c) applying the same bending moment on the two slab ends under simply supported conditions, to get the equivalent bending $E_{\text{bend, y}}$ by making them have the same vertical displacements at the middle span; (d) applying a linear temperature gradient, with 13°C to -13°C (23°F to -23°F) from the top surfaces to the bottom and under the X DOFs fixed support conditions, to obtain the $\alpha_{\text{bend, x}}$ by making the two slabs have the same bending moments. Therefore, Table 7-1 listed the properties of the original GFRP slab and the proposed properties of the equivalent slab for both the pure axial and bending loading conditions.

![Fig. 7-4 Represenative GFRP Slab and Equivalent Slab](image)

### Table 7-1 Properties of GFPR Slab and Equivalent Slab

<table>
<thead>
<tr>
<th>Property</th>
<th>GFRP Slab</th>
<th>Equivalent Slab</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>Facial</td>
<td>Core</td>
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<tr>
<td>$E_x$ (psi)</td>
<td>2.85E+06</td>
<td>1.71E+06</td>
</tr>
<tr>
<td>$E_y$ (psi)</td>
<td>1.85E+06</td>
<td>1.71E+06</td>
</tr>
<tr>
<td>$E_x$ (psi)</td>
<td>1.85E+06</td>
<td>1.71E+06</td>
</tr>
<tr>
<td>$\alpha_x$ (L/L°F)</td>
<td>6.77E-06</td>
<td>1.14E-05</td>
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<tr>
<td>$\alpha_y$ (L/L°F)</td>
<td>1.11E-05</td>
<td>1.14E-05</td>
</tr>
</tbody>
</table>

*Note:
1) The properties of facial and core laminates are referred from Oghumu (2005)
2) The properties in Z direction and shear modulus are not provided since they are insignificant when considering pure bending and axial deformation conditions.
7.3.3 Verification of GFRP Panel Properties

The thermal responses of the two GFRP and EQUIV slabs with a much larger size of 2.4m×3.6m×0.13m (96in×140in×5in) are used for the verifications of the proposed properties. Table 7-2 and Table 7-3 list the comparisons of the internal forces at the middle span of the two slabs, where the supports are fixed in the X and Y axes directions, respectively, and the slabs are subjected to a gradient temperature distribution with 25.6°C (46°F) and 0°C (0°F) from the top to the bottom surfaces. It can be observed that the summation of the uniform and linear temperature loading results in the EQUIV slab are matching well with that of the original GFRP one, where the differences of the induced thermal forces are negligible for the axial forces and are less than 10% for the bending moments. In addition, it is verified that the final superposition results in the EQUIV slabs are almost exclusively contributed by the uniform and gradient temperature loading components. Moreover, Figure 7-5 and Figure 7-6 show two representative cases for both slabs being subjected to the 13°C (23°F) pure linear temperature gradient under the X and Y DOFs fixed supports, respectively, where the vertical displacement contour maps are matching well between the two GFRP and EQUIV slabs. Therefore, the proposed equivalent slab properties, both the axial and bending ones, are equivalent to those of the original GFRP panels.

<table>
<thead>
<tr>
<th>SLAB</th>
<th>Load</th>
<th>$F_x$ (lb)</th>
<th>$M_y$ (lb-in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GFRP</td>
<td>Linear Gradient (46°F to 0°F)</td>
<td>-65723.68</td>
<td>-157927.50</td>
</tr>
<tr>
<td>EQUIV</td>
<td>Uniform (23°F)</td>
<td>-65737.86</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>Linear Gradient (23°F to -23°F)</td>
<td>-0.02</td>
<td>-144333.50</td>
</tr>
<tr>
<td></td>
<td>Sum</td>
<td>-65737.88</td>
<td>-144333.48</td>
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<tr>
<td>Comparison</td>
<td>Difference (%)</td>
<td>0.02</td>
<td>-8.61</td>
</tr>
</tbody>
</table>

**Table 7-2** Comparisons between GFRP and EQUIV Slab with the X- Dir. Supports Fixed

<table>
<thead>
<tr>
<th>SLAB</th>
<th>Load</th>
<th>$F_y$ (lb)</th>
<th>$M_x$ (lb-in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GFRP</td>
<td>Linear Gradient (46°F to 0°F)</td>
<td>-40379.24</td>
<td>85437.43</td>
</tr>
<tr>
<td>EQUIV</td>
<td>Uniform (23°F)</td>
<td>-40493.06</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>Linear Gradient (23°F to -23°F)</td>
<td>0.02</td>
<td>79739.25</td>
</tr>
<tr>
<td></td>
<td>Sum</td>
<td>-40493.04</td>
<td>79739.25</td>
</tr>
<tr>
<td>Comparison</td>
<td>Difference (%)</td>
<td>0.28</td>
<td>-6.67</td>
</tr>
</tbody>
</table>
7.4 Numerical Study Results

7.4.1 IAB Project Study- Base Case

A field monitoring program was conducted on the Caminada Bay IAB, LA, over one year since 08/11/11, in which a large amount of instrumentations were installed to measure the bridge responses due to the temperature variations. Figure 7-7 shows the measured temperatures at surfaces of the bent top, approach slab bottom, and deck slab bottom. Figure 7-8 shows the measured hourly-varying temperatures of the slabs and ambient during the hottest week from 08/19/11 to 08/26/11. According to the approaches provided by the AASHTO LRFD (2007) specifications, together with the considerations of the two extreme air temperatures, 4.4°C (40°F)
and 29.4°C (85°F) during the monitoring period, the effective bridge temperature that will induce seasonal movements for this IAB is calculated as 37°C (68°F), and the maximum positive temperature gradient is observed appearing on 08/20/11 17:00 pm with a difference of 10°C (18°F).

Fig. 7-7 Measured Hourly Varying Temperature of Bridge Slab

Fig. 7-8 Bridge and Air Temperature during the Hottest Week from 08/19/11 to 08/25/11
These measured temperatures are applied as node forces and incorporated into the finite element model during the following analysis. Additionally, since the support conditions show significant effects on the bridge behaviors of IABs, two extreme cases with both fully free and fully fixed conditions are considered in the numerical study. Therefore, the thermal responses in terms of the Bent1 displacements, slab strains, backfill pressures, and y-axis bending moments are plotted from Figures 7-9 to 7-12, respectively, where Bent1 is shown as the location A in Figure 7-1. For the convenience of discussions, the legend in each plot is uniformly defined here by its loading and support conditions, i.e., U and G refer to the uniform and gradient temperature loadings, respectively. For example, the legend of the G_Fixed Support means the case when the bridge is subjected to gradient temperatures under the rigidly supported condition.

Based on the observations of these figures, some of the conclusions can be drawn as follows. First, the case with free supports and uniform temperature variations induces most significant effects on the bridge performances. It causes larger displacements, and that in turn, generates higher backfill pressures on the abutment and greater positive and negative bending moments along the pile. Second, the support conditions, together with the temperature loading types, apparently affect the thermal strains that generated in the slabs. For example, for the uniform temperature increase case, the significantly higher compressive strains are appearing when the bridge movements are fixed; while for the gradient distribution condition, positive strains are shown at the slab bottom surfaces under the free supports condition. In addition, the most critical conditions for slabs are the cases with fixed supports and gradient temperature loadings. Third, the slab strain distributions along the bridge length direction show no differences between the locations at the bridge ends and middle parts even though larger responses should have appeared near the Bent1 due to the structure integration and backfill restraints at that location. This phenomenon may be attributed to the special structure configuration of this IAB that all the interior slabs are fixed on the interior bents, and the backfill effects on the abutment are negligible.

The results from the study of this project case indicate that it is not advisable to allow too much deformation on the superstructure for the safety of the substructure and soil, especially if designing longer span bridges with larger thermal displacements. However, if the movements of the superstructure are restraint, e.g., due to the fixed supports or much stiffer backfill materials, it
could generate greater internal forces on the slabs. In this sense, it can be concluded that, the total length of the bridge cannot be elongated unless the induced thermal movements are appropriately accommodated either by the superstructure, substructure, or both. Therefore, the following sections of this paper attempt to discuss the condition when the thermal movements or forces are mostly absorbed and sustained by the superstructure. Then, the thermal responses of the substructure are assumed not to be under a critical condition but meeting the design requirements. Specifically, the concrete slabs are replaced by the GFRP panels using the homogenized slab and the equivalent material properties proposed above, and the corresponding thermal responses on the slabs are discussed to investigate whether such a configuration would provide any benefits.

Fig. 7-9 Bent1 Displacements under Different Temperature Loadings and Support Conditions

Fig. 7-10 Slab Strains under Different Temperature Loadings and Support Conditions
7.4.2 Verification of Stiffness-Equivalent Method

In addition to the discussion above, where the predicted material properties and proposed stiffness-equivalent modeling method is verified by studying a simple case, i.e., a structure with the panel alone under the gradient temperatures and fixed supports, the rationale of this method is again justified by studying an actual bridge including the superstructure, substructure, and soil. Specifically, the calculated internal forces, at the location shown by the label C in Figure 7-1, for the base case of IAB with concrete slabs under free and fixed support conditions are listed in
Table 7-4 and Table 7-5, respectively. In each table, the first column lists the results from the method, designated as I, that directly applying a gradient temperature of 25.6°C (46°F) and 0°C (0°F) on the slab. In addition, the results, that using the stiffness-equivalent approach by first decomposing the temperatures into two components, i.e., uniform increase with 12.8°C (23°F) and linear gradient with 12.8°C (23°F) and -12.8°C (-23°F), and then superposing them together afterwards, are listed in column 2 to 4, respectively, and that is designated as method II.

Based on the observations of the results in the tables, it can be found that the summation results from II are matching well with that from I, especially for the bridge under free support conditions. For example, the axial and bending stresses in I are, to a great extent, contributed by about 91% and 95% of the uniform and gradient temperature loading results from II. For the fixed support condition, however, it is not the case where the results from the uniform loading component in II contribute about one third of the total bending stress in I, even though the axial stress in I are still primarily coming from the uniform loading component in II. This phenomenon is attributed to the special feature of the IAB configuration. The bridge is not a determinate but indeterminate structure, which is rigidly connected between the slabs and bents, restrained by the soils behind the bents and surrounding the piles, and fixed supported at its two ends. Then, when subjected to the pure uniform temperatures, the slab will both translate and rotate; thus both the axial forces and bending moments are generated when these deformations are restrained by the structural redundancies. However, this effect is not significantly high, and it is only appeared in the fixed support condition. Also, the final results are still to a great extent contributed by both the pure uniform and gradient temperature loading components, respectively. Then, this simplified stiffness-equivalent method is still valid and will be adopted in the following analysis of the GFRP panel bridges.

<table>
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<tr>
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<tbody>
<tr>
<td>Fx (lb)</td>
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<td>2.20E+04</td>
<td>3.05E+05</td>
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<td>5.31E+05</td>
<td>2.01E+07</td>
<td>2.07E+07</td>
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<td>Axial Stress (psi)</td>
<td>28.28</td>
<td>25.75 (91.05%)</td>
<td>2.00 (7.07%)</td>
<td>27.75 (98.12%)</td>
</tr>
<tr>
<td>Bend Stress (psi)</td>
<td>638.91</td>
<td>16.11 (2.52%)</td>
<td>610.90 (95.62%)</td>
<td>627.01 (98.14%)</td>
</tr>
</tbody>
</table>

Table 7-4 Forces at Slab Section under Free Support
Table 7-5 Forces at Slab Section under Fixed Support

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Fx (lb)</td>
<td>5.03E+06</td>
<td>5.03E+06</td>
<td>-9.49E+04</td>
<td>4.93E+06</td>
</tr>
<tr>
<td>My (lb-in)</td>
<td>2.98E+07</td>
<td>9.27E+06</td>
<td>1.99E+07</td>
<td>2.92E+07</td>
</tr>
<tr>
<td>Axial Stress (psi)</td>
<td>457.49</td>
<td>457.68 (100.04%)</td>
<td>8.64 (1.89%)</td>
<td>466.31 (101.93%)</td>
</tr>
<tr>
<td>Bend Stress (psi)</td>
<td>902.68</td>
<td>281.26 (31.36%)</td>
<td>604.93 (67.02%)</td>
<td>886.20 (98.16%)</td>
</tr>
</tbody>
</table>

7.4.3 GFRP Panel Applications

A parametric study is conducted in this section to investigate the thermal responses of IABs by replacing the concrete slabs with GFRP panels, and some of the parameters are defined as follows. First, the original isotropic concrete slab is still considered as the base case with the properties of $E_x=3.95E+06$ psi, $\alpha_x=6.00E-06$ L/L/ºF; and the properties of the orthotropic GFRP slabs in the two main axes directions are separately considered in the model by aligning X and Y axes with the longitudinal direction of the bridge, respectively. Second, two temperature design loadings, referred to AASHTO LRFD (2007), are assumed to be applied on both slab cases. This assumption may somehow underestimate the maximum seasonal and daily temperature variations of the FRP bridges, since these recommended values in the code are primarily specified for concrete and steel bridges; and the authors of this paper have already reported elsewhere that the temperature distributions of the GFRP panel bridges may be about 30% higher than that of the concrete ones due to the different heat transferring coefficients (Kong and Cai 2012c). Since no experimental data or other code information available, the temperature variations with a 44.4ºC (80ºF) uniform increase and a 25.6ºC (46ºF) gradient are assumed for both slabs at this stage, and the effects of this assumption on the final conclusions will be proven unimportant in the following discussions. Third, two extreme support conditions with both fully rigid and fully released are adopted again. Last, for the connection details between the GFRP panels and bents, they are assumed as fixed conditions similar to that of the adopted in the IAB project. This assumption should provide more critical results compared to the actual practice. In real constructions, the connections are often either using adhesives or clamping with devices, in which case, certain movements or rotations may be allowed and non-composite or partial-composite behaviors are often provided. Thus, the induced forces will be smaller than that of the fully-composite condition as assumed here. In addition, the legend for each parametric case is designated in three parts, i.e., material types (C refers to concrete, Gx and Gy refers to the GFRP slab with its X and Y main axes material properties along the bridge length, respectively),
loadings (80U refers to uniformly increase of 44.4 °C (80°F), and 46G refers to a linear distribution of 25.6°C (46°F)), and support conditions (F refers to free and R refers to rigid). For example, Gy_46G_R is the case with a 25.6°C (46°F) linear temperature distribution on the GFRP slab with its Y-direction properties along the bridge length and under the fixed support conditions at the two bridge ends.

Figure 7-13 and Figure 7-14 compare the stresses at the bent top and slab bottom surfaces, shown at the B and C locations in Figure 7-1, between IABs with the concrete and GFRP slabs, respectively, in which the complicated slab responses are observed under the current AASHTO LRFD (2007) temperature loadings and different support conditions.

First, it can be found that the gradient temperatures generally produce larger stresses under the free supports while the uniform temperatures generate larger ones under the fixed supports. In addition, opposite stresses signs are appearing at the slab bottom surfaces under current positive temperature variations. If the negative temperature variations are considered, or the temperature loadings are superposed with other loading types, the positive stresses may be generated at the slab top surfaces, in which case, the concrete may experience crack failures due to their lower tensile strength capacities. For GFRP panels, however, strength failures are no longer a big issue. Table 7-6 lists the tensile and compressive strengths for a series of commonly utilized GFRP materials that are applied in practice. Through comparisons, the tensile and compressive strengths of GFRP materials are at least more than 5 times larger than that of the concrete, e.g., the bridge in this study adopts the Class AA (M) concrete with a compressive strength of 28Mpa (4000psi).

Second, compared with the concrete slab case results, the stresses induced in the GFRP panel cases, either with X or Y axes material properties along with the bridge length direction, are larger under the uniform temperature loading and free support cases but smaller under gradient loading ones. This scenario is the results of the synthesized effects from multiple factors, including the section rigidities, material thermal expansion coefficient, and the support restraint conditions. However, no matter in either case, the stresses induced in the GFRP panels under current structural configurations are not significantly different compared to that in the concrete slabs; and the stresses percentages that accounting for the total material strength capacities are far smaller in GFRP panels than that in concrete slabs.
Third, another concerning aspect of the applications of FRP panels is that large vertical deformations are possibly provided due to their smaller section rigidities. According to AASHTO LRFD (2007), it requires that the calculated deflections, if using a live load distribution factor, do not exceed span length/800, or span length/1000 for pedestrian sidewalks, respectively. In this study, however, the induced vertical displacements, if only considering the designed temperature loadings from AASHTO LRFD (2007), are far smaller than the empirical values. In addition, in most IABs, especially the one adopted in this study, the bridges are often with much rigidities due to the extra stiffness provided by the structural integration and soil restraints; thus IABs should be beneficial for the FRP panels and to help reduce the vertical deformations.

Therefore, replacing concrete slabs by GFRP panels in IABs brings satisfying thermal performance as is expected. The structures can be designed in a way that the thermal movements are primarily accommodated and absorbed by the superstructure. In this sense, the movements transferred to the substructures are limited and smaller forces will be expected to exert on piles and soils. For the superstructure, under the temperature loadings, the induced stresses on GFRP panels will only account for a small percentage of their larger tensile and compressive strength capacities. Thus, more extreme environmental temperatures are expected to be accommodated and much longer span bridges are to be designed for IABs with FRP panels.

![Bent Top Surface Stresses](image)

**Fig. 7-13** Bent Top Surface Stresses on (a) Free Support and (b) Fixed Support
Fig. 7-14 Slab Bottom Surface Stresses on (a) Free Support and (b) Fixed Support

Table 7-6 Strength of GFRP materials (http://en.wikipedia.org/wiki/Fiberglass)

<table>
<thead>
<tr>
<th>Material</th>
<th>Tensile MPa (ksi)</th>
<th>Compressive MPa (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Polyester and chopped strand mat laminate 30% E-glass</td>
<td>100(14.5)</td>
<td>150(21.8)</td>
</tr>
<tr>
<td>Polyester and woven roving laminate 45% E-glass</td>
<td>250(36.3)</td>
<td>150(21.8)</td>
</tr>
<tr>
<td>Polyester and satin weave cloth laminate 55% E-glass</td>
<td>300(43.5)</td>
<td>250(36.3)</td>
</tr>
<tr>
<td>Polyester and continuous roving laminate 70% E-glass</td>
<td>800(116)</td>
<td>350(50.8)</td>
</tr>
</tbody>
</table>

7.5 Conclusion

Integral abutment bridges (IABs) are gaining wide acceptances during the recent several decades due to their satisfying performances. However, some arguments are still unsettled on them. One of the several challenges is to find an appropriate approach to accommodate the thermal movements from the superstructure. The current adoptions of the concrete superstructure may limit the maximum length that can be designed since the concrete materials are with relatively larger thermal expansion coefficients but smaller tensile strength capacities. One innovative idea is proposed and justified in this study to replace the concrete slab with a new GFRP panel since the latter one has much higher strength capacities and its material properties are able to be designed. Therefore, based on the investigations, some of the conclusions are drawn as follows.
(1) The proposed homogenization method, by first decomposing and then superposing the loadings and results into two uniform and gradient temperature components, is appropriate in the study of this complicated honeycomb, sinusoidal, hollowed section, and sandwich GFRP panel. In addition, the proposed material properties under both pure axial and bending loading cases are justified by a slab example subjected to gradient temperature loadings.

(2) When incorporating the GFRP panel into the full IAB model, however, the stiffness-equivalent method may not be completely appropriate, since the structural redundancies will provide both of the axial and bending forces even only the uniform temperatures are applied. However, this phenomenon primarily appears in the fixed support conditions, and the induced bending forces under uniform temperature loading cases are not significantly high. Thus, the homogenization and stiffness equivalent method can still be adopted in the analysis of the IAB thermal performance with GFRP panels.

(3) Through studying the responses of the Caminada Bay IAB with the measured uniform and gradient temperature loadings, it indicates that allowing too much deformation on the superstructure will generate larger backfill pressures and pile forces. However, if restraining these thermal movements, significantly higher thermal stresses will be induced in the slabs. In addition, the slab thermal stresses are found varying with the change of the loading types and support conditions, where the most critical conditions for slabs are appearing in the cases with fixed supports and gradient temperature distributions.

(4) Benefits are observed when replacing the concrete slabs by GFRP panels on IABs. The stresses failure issues, especially the tensile cracks that often appeared in the concrete slabs, are no longer critical in GFRP panels, because both the tensile and compressive strength of the GFRP materials are significantly higher than that of the concrete. In addition, the induced vertical displacements under temperature loadings are also within the recommended ranges that specified for the live load design by AASHTO LRFD (2007). Moreover, the special features of IABs, e.g., structure integration and soil restraints, will help control the vertical displacements of GFRP panels.

(5) In summary, based on the tentative numerical investigations, the applications of GFRP panels on IABs show satisfying performance for both the superstructure and substructure. In the
future, it is possible to extend the length of IABs or adapt the IABs to other complicated soil or structure conditions, as long as the material properties of the FRP panels are designed to meet different physical or mechanical requirements under the temperatures, traffics, or other loadings.

7.6 References


167


CHAPTER 8. CONCLUSIONS AND RECOMMENDATIONS

Using the field monitoring study method and numerical simulation method, the thermal behaviors of IABs and FRP panel bridges are first investigated. Then, the issues and benefits, after replacing the traditional slabs with FRP panels for both the jointed and integral bridges, are discussed and verified. Some of the conclusions can be drawn as follows:

8.1 Thermal Behavior of FRP Panel Bridges

Based on the field monitoring program conducted on the GFRP honeycomb sandwich panel at the state of Kansas, it can be concluded:

(1) Significant temperature differences are observed along the panel depth of the GFRP panel, and that is attributed to the special thermal transferring characteristics of FRP panels such as the hollowed section configurations and lower thermal conduction abilities.

(2) The current temperature design specification in AASHTO LRFD (2007) code, originally specified for the traditional concrete slab bridges, is no longer valid for GFRP panels since the measured temperatures have already exceeded the range that is stipulated for concrete slabs in the code.

(3) The heat transfer mechanisms within the hollow section, i.e., the inner air convection and mutual radiation, are proven to produce comparatively less effects on the temperatures generated at the panel surfaces but apparent influences on the thermal gradients through the panel depths. Thus, a neglect of the section hollowness may underestimate the magnitude of temperature effects.

(4) The daily maximum ambient temperature, solar radiation, and wind speed are proven to be the primary factors determining the temperature distribution patterns of the GFRP panels; while the material thermal properties only influence the thermal gradients to a small extent.

Based on the numerical study on the composite bridges with FRP panel slabs, the thermal gradients for designs have been proposed by improving and correcting the available design specifications of the concrete and steel bridges in AASHTO LRFD (2007).
(1) For the temperature distributions of the bridges with GFRP slabs and concrete beams, three segmental linear distributions are proposed including one linear segment for the region from the GFRP panel’s top surface to the top of the concrete beam, one linear segment the same as that specified in AASHTO LRFD (2007) code near the concrete beam bottom, and one constant segment with a zero gradient through the rest beam depth. In addition, as for the temperature values at the GFRP panel’s top surface, they need to be determined according to the actual layout of materials and their corresponding solar absorptivity.

(2) For the temperature distributions of bridges with GFRP slabs and steel beams, two segmental linear distributions are defined with one linear segment for the region from the GFRP panel’s top surface to the steel girder’s top surface, and a constant temperature distribution, with a larger value compared to that specified in the AASHTO LRFD (2007) code, through the rest steel girders. Also, the temperature values at the panel’s top surfaces are determined based on the solar absorption abilities of the surface materials.

(3) Based on the predicted temperature distributions, the thermal strain distributions of the FRP bridges are observed and the large thermal stresses are generated at the interfacial locations between the FRP slabs and concrete or steel beams.

For the bridge responses, i.e., forces and deformations, after replacing traditional slabs with FRP panels under the temperature, live load, and dead load conditions, it can be concluded:

(1) For the slab replacement projects conducted by KSDOT, where steel decks were replaced by the GFRP panels, the induced thermal responses, simulated using the measured nonlinear thermal gradients and uniform temperature variations from January 24 to July 13, 2004, are larger compared to that from the effects of the dead load and HS-20 live load, even though the deflections in all cases are still within the AASHTO LRFD (2007) requirements and the generated thermal stresses are only taking a small portion of the materials’ strength capacity.

(2) For the general conditions of slab replacements by FRP panels, the deflections after slab replacements are increasing compared to the original concrete slab cases, and these deflections are larger than that caused by the live load and dead load effects.
(3) Different from the traditional concrete and steel bridges, where vertical deflections can hardly be generated under the uniform temperature variations, the GFRP panel bridges could still have large deflections under such loading conditions due to the distinctive material thermal properties at the interfacial locations between the GFRP panels and the concrete or steel beams.

(4) Even though the GFRP panels have distinctive thermal expansion coefficients, yet the induced horizontal movements, after slab replacements, are still mainly contributed by the deformations of the steel girders or concrete beams since the GFRP panel has comparatively smaller geometric sizes.

(5) Thermal stresses are induced due to the nonlinear thermal gradients and the material property differences at the interfaces between two different materials. It has been observed that, by replacing concrete slabs with GFRP panels, the induced thermal stresses are evidently increased especially at the top surfaces of the girders. This phenomenon should be carefully considered, since the thermal stresses are already comparable and even exceeding the effects from that of the dead load and live loads.

8.2 Field Monitoring Study of Integral Abutment Bridges

Based on the field measurement of the first full integral abutment bridge, Caminada Bay Bridge on soft soil conditions, over one year from 08/11/11 to 08/11/12, some important observations can be concluded as follows:

(1) The periodical variation trends are observed from the measured environmental air temperatures, and they can be adopted to predict the bridge temperatures by curve fitting methods. In addition, the measured wind speeds indicate the highly-varying thermal responses at the surfaces of the slabs.

(2) The measured seasonal and daily slab temperatures are almost reaching or exceeding the maximum design values that are specified in the AASHTO LRFD (2007). It verifies that the primary temperature variations within the bridge are exclusively appearing in the superstructure and that further induce the bridge thermal responses.

(3) Both the strains measured at the bent top surfaces and slab bottom surfaces show a good correlation with the temperature variations. The tension stresses appeared at the bent top
surfaces may crack the concrete, while the compressive stresses are negligible. In addition, there
is no difference for strains induced at the locations between the bridge ends and middle spans
under such a bridge with soft soil and stiff structure configurations.

(4) Soils behind the abutment affect the behaviors of the integral abutment in terms of its
displacements and rotations. These effects are complicated and the soil restraints on the abutment
deformations are accumulated with time due to the plastic behaviors of soil. However, for the
soils away from the abutment, or when the connection details between the slab-bent are not
integral, these soil effects are negligible.

(5) The bending strains of piles, with respect to the two main axes, are both important due
to the bridge skew effects and rigid pile-bent connections for the current bridge case, even
though the strain values are only taking a small portion of the material compression capacity. For
the pile bending profile induced from the slab thermal movements, double curvatures are
observed and the zero moment point is located approximately at one third of the pile length from
the pile head.

8.3 Numerical Modeling Study of Integral Abutment Bridges

Based on the numerical study of the general integral abutment bridges under different
structural and geotechnical parameters, some of the important conclusions are drawn as follows:

(1) Bridges with fully free horizontal supports at the superstructure induce larger bridge
movements, and those in turn, cause greater backfill pressures and pile internal forces; while the
fully fixed support conditions on the superstructure induce the bigger thermal stresses in the
slabs even though such a support configuration is beneficial for the substructure.

(2) The backfill effects on the bridge are negligible under the current structural geometries
and configurations, due to the shallower height and soft soil types. However, in general
conditions, the stiffer backfill soils produce bigger internal forces on the superstructure; and the
softer backfill soils, otherwise, can generate larger forces on the substructure.

(3) The soils surrounding the piles show the most obvious effects on the bridges. If
changing the soft soils to the stiff ones, it will provide a maximum of 1.5 times smaller bridge
displacements and 20% smaller backfill pressures; but at the same time, it will also induce 70% larger pile positive strains and 48% larger slab negative strains.

(4) The connection behaviors between the pile-bent can affect the responses at the piles and abutments. They will change the locations of the maximum pile internal forces. They also can affect the rotation behaviors of the abutment, and further influence the backfill pressures and slabs strains to some extent.

8.4 Applications of FRP Panels on Integral Abutment Bridges

Through the investigations on the responses of Caminada Bay IAB based on the measured uniform and gradient temperature loadings, it can be concluded as:

(1) The slab thermal stresses are found varying with the change of the loading types and support conditions. The most critical conditions for slabs are observed appearing in the cases with fixed horizontal supports and gradient temperature distributions.

(2) Benefits are observed when replacing the concrete slabs by GFRP panels on IABs. The stress failure issues, especially the tensile cracks that often appeared in the concrete slabs, are no longer critical in the GFRP panels, because both the tensile and compressive strength capacities of the GFRP materials are significantly higher than that of the concrete.

(3) The induced vertical displacements under temperature loadings are within the recommended ranges that are specified for the live load designs in AASHTO LRFD (2007). In addition, the special features of IABs, e.g., structure integrations and soil restraints, will help control the vertical displacements of GFRP panels.

(4) Generally speaking, based on the tentative numerical investigations, the applications of GFRP panels on the IABs show the satisfying performance for both the superstructure and substructure. More importantly, in the future, it is possible to extend the length of IABs or adapt the IABs to other complicated soil or structure conditions, as long as the material properties of the FRP panels are designed to meet the corresponding requirements when considering the temperatures, traffics, soils, or other loading conditions.
8.5 Recommendations for Future Research

Both the construction of IABs and the application of FRP panels are relatively new topics in the bridge engineering. For example, no national design guidelines exist for IABs and the current designs in each state are primarily relying on empirical practice. Similarly, the FRP panels are mostly used for the rehabilitations and retrofits of deteriorated structures. However, the applications of FRP panels on the completely new bridges, either the jointed or integral ones, are still very few. Therefore, many meaningful aspects exist for research in the future.

For integral abutment bridges, due to the structure integration and the soil incorporation, the static and dynamic properties of the whole bridge will be different compared to the traditional jointed ones. In this case, some of the design criteria in AASHTO LRFD (2007) may no longer be valid and need to be improved and corrected, such as the live load distribution factor, dynamic allowance factor, allowable design temperature or movement, etc. In addition, most of the current studies are more focused on the research stages, where the performance of IABs under different soils, structures, and loadings are investigated using the complicated numerical modeling methods. Therefore, some simplified and convenient approaches for the routine designs are needed. Finally, one of the most challenging tasks is to figure out the ways to extend the length of the IABs and to fully appreciate the benefits of such jointless bridges. Even though one method, using the FRP panels to replace concrete slabs, is proposed in this study, it should also try some other approaches, such as the FRP piles, stress relieving mechanisms for soils, innovative structural configurations and materials, etc.

For FRP panel bridges, a lot of efforts have been put into the study of their static and dynamic behaviors; while it still requires more in-depth investigations on their thermal responses. This research analyzed the FRP panel bridges in the global perspective of view, such as the global thermal deformations, strains, and stresses of the bridge. However, it is also meaningful to study some local behaviors, such as the delamination and buckling failures induced by thermal effects, since these failures are also often observed in the field tests. In addition, for the idea of applying FRP panels on IABs, as is stated in this study, it is all new with only one or two actual applications based on the authors’ knowledge. Thus, a lot of uncertainties still exist. This research demonstrates and verifies their possible benefits in resolving the thermal issues of IABs using the assumed material and structure properties, but many other aspects deserve further
investigations. Finally, one of the most important aspects of applying FRPs in bridge engineering is to design specific material properties for certain requirements. Thus, it should propose some simplified design method for FRP structures and then investigate the feasibilities of their applications in bridges.
VITA

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