

Spatial and Temporal Load Distribution in Steel Bridge Superstructures (Vol. III): Investigation of Skew and Abutment Influence on Steel I-Girder Bridge Performance under Live Load and Long-Term Thermal Effects through Field Data and Parametric Studies

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16. Abstract Skewed steel I-girder integral abutment bridges (IABs) are increasingly used due to their structural efficiency and elimination of deck joints. Thermally induced behavior of such bridge systems, however, remains insufficiently understood. This report documents the live load and thermal behavior of skewed steel I-girder bridges, which are representative of current Illinois bridge practice, through long-term field monitoring and numerical parametric studies. Three-dimensional finite element models developed in ABAQUS showed good agreement with field-measured thermal response and highlighted the importance of realistic representation of support restraint and soil-structure interaction effects. Live load parametric studies demonstrated that skew angle, support restraint, cross-frame configuration, and bottom flange unrestrained length strongly influence flange lateral bending stresses, with significant stresses (up to 28 MPa [4 ksi] on interior girders) observed even below current AASHTO and IDOT skew thresholds. Long-term field monitoring further demonstrated that substantial thermally induced stresses develop in skewed steel I-girder bridges, particularly in IAB configurations, where lateral bending stress ranges up to 30 MPa (4.3 ksi) were observed near the abutment region on an exterior girder. Thermal parametric studies showed that thermally induced lateral bending stresses increase substantially with skew angle (from approximately 13–21 MPa [2–3 ksi] at low skew angles to 38 MPa [5.5 ksi] at high skew angles) and integral support restraint, with the largest stresses concentrated near abutment and pier regions. Overall, the results suggest that stresses due to thermal effects may warrant explicit consideration in the design and evaluation of skewed steel I-girder IABs.					
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EXECUTIVE SUMMARY

Skewed steel I-girder integral abutment bridges (IABs) are increasingly used due to their structural efficiency and elimination of deck joints. Thermally induced behavior of such bridge systems, however, remains insufficiently understood. This research investigated the live load and thermal behavior of skewed steel I-girder bridges through detailed numerical modeling, long-term field monitoring, and numerical parametric studies using bridges that are representative of current Illinois bridge practice. Three-dimensional finite element (FE) models were developed in ABAQUS to simulate bridge response under live and thermal loading, explicitly representing girders, decks, cross-frames, bearings, piers, abutments, and soil-structure interaction effects, including cyclic p - y behavior for integral abutment pile foundations. Validation studies demonstrated good agreement between FE-predicted and field-measured response under long-term thermal loading and highlighted the importance of realistic representation of support restraint and boundary conditions when evaluating thermally induced bridge behavior.

Live load parametric studies demonstrated that skew angle, support restraint, cross-frame configuration, and bottom flange unrestrained length strongly influence development of flange lateral bending stresses. Significant lateral bending stresses (up to 28 MPa [4 ksi] on interior girders) were observed on IABs even below the AASHTO applicability threshold of 20° skew and below the 45° threshold currently adopted by the Illinois Department of Transportation (IDOT) for explicit consideration of lateral bending stresses. Interior girders were generally more critical than exterior girders regarding live load-induced lateral bending behavior, while X-cross-frame systems produced substantially larger stresses than channel configurations.

Long-term field monitoring of the two two-span skewed steel I-girder bridges—one with integral abutments and one with stub abutments—demonstrated that substantial thermally induced stresses develop in skewed steel I-girder bridges, particularly in IAB configurations. Maximum recorded weak-axis bending stress ranges reached approximately 30 MPa (4.3 ksi) near abutments and 18 MPa (2.5 ksi) near midspan in the monitored IAB (both on exterior girders), while the monitored stub abutment bridge reached approximately 17 MPa (2.4 ksi) on an exterior girder. Approximately one-half of these ranges would develop in a single direction at a given time. Axial stresses were governed primarily by long-term thermal expansion and contraction, whereas strong-axis and weak-axis bending stresses exhibited substantial short-term deviations associated with thermal gradients.

Thermal parametric studies demonstrated that thermally induced lateral bending stresses increase substantially with skew angle and integral support restraint. While axial stresses remained relatively stable, weak-axis bending stresses increased from approximately 13–21 MPa (2–3 ksi) at low skew angles to approximately 38 MPa (5.5 ksi) at high skew angles (both stub and integral abutment bridges), with the largest stresses concentrated near abutment and pier regions. Comparisons with current AASHTO recommendations indicated that thermally induced lateral bending stresses alone can represent a substantial portion of the stress levels currently recognized for skew-related lateral bending behavior under live and dead loads. Furthermore, meaningful thermal lateral bending stresses developed even below the 20° AASHTO applicability threshold and the 45° skew threshold currently adopted by IDOT, suggesting that thermal lateral bending effects may warrant explicit consideration in skewed steel I-girder IABs.

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CHAPTER 1: INTRODUCTION

Illinois Center for Transportation project R27-194 aims to provide a comprehensive understanding of load distribution in skewed steel I-girder bridges and to potentially refine their analysis and design procedures based on results from field monitoring and numerical simulations. This report, Volume III, expands on the live load analysis carried out in Volume II (Zhou et al. 2026a). Additionally, the long-term collected data are presented and evaluated, and numerical simulations are used to understand parameters influencing integral steel I-girder bridge response under thermal loading. The background of this research project and details regarding field monitoring and numerical simulation approaches are elaborated in Volume I (Zhou et al. 2026b). Additional features of the 3D finite element (FE) models specifically implemented for long-term (thermal) models are discussed in this report.

SUMMARY OF FIELD MONITORING

To facilitate a greater understanding of skewed steel I-girder bridge behavior under construction dead and in-service live loads, two adjacent bridges in Champaign, Illinois—I-74 and I-57 over Mattis Avenue (referred to as Mattis-74 and Mattis-57, respectively)—were monitored and studied. The instrumentation plans of both bridges are shown in Figures 1 and 2, respectively. Due to their close proximity, both bridges are subjected to nearly identical environmental conditions and similar traffic. Mattis-74 is a 41° skew stub abutment bridge with 12 steel I-girders (1.8 m [70 in.] web height), while Mattis-57 is a 45° skew integral abutment bridge (IAB) with 8 steel I-girders (1.9 m [74 in.] web height). Both bridges are two-span continuous with X-shaped cross-frames arranged perpendicular to the girder lines in a staggered configuration. The field-monitoring program collected strain variations at select steel I-girders cross-sections, cross-frames, and concrete locations; rotations of select girders (using tiltmeters); and relative superstructure-substructure movement (using displacement transducers, or crackmeters), as shown in Figures 1 and 2.

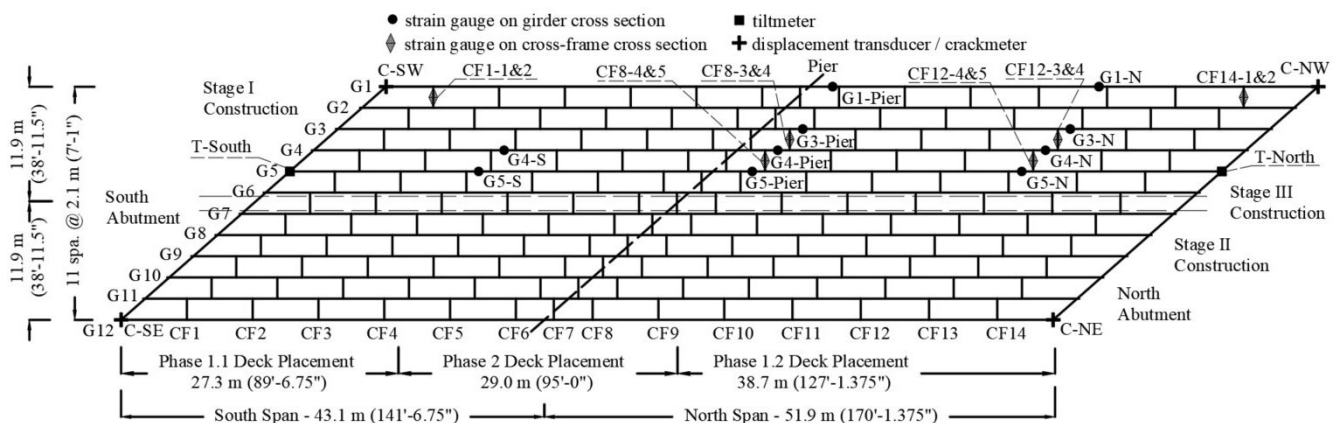


Figure 1. Plan view. Mattis-74 with instrument locations.

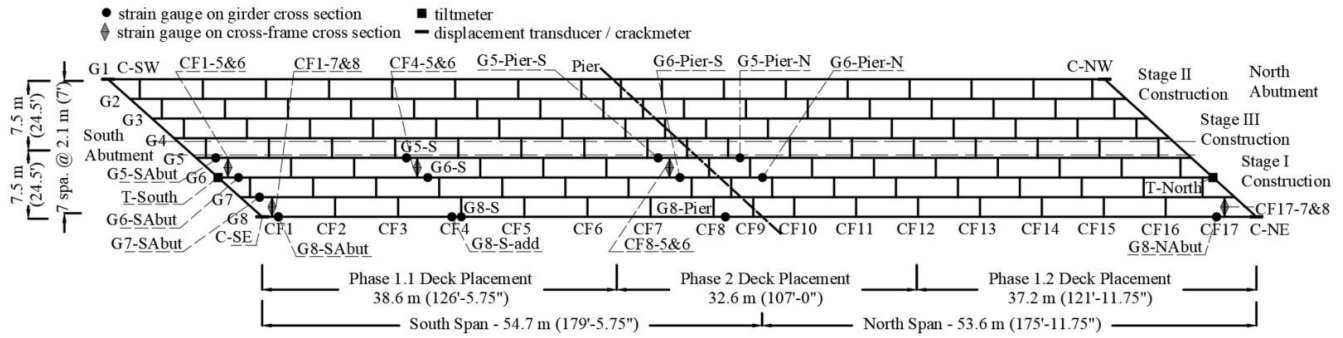


Figure 2. Plan view. Mattis-57 with instrument locations.

Data collection started before the bridges were in service (December 2020) and continued until December 2025. Behavior of the monitored bridges during construction stages and early in-service stages is presented in Volume II (Zhou et al. 2026a). The field data presented and discussed in the current report spans from September 2021 until the end of the monitoring period (December 2025).

Girder cross-sections were either heavily instrumented—with eight strain gauges on the girder and one embedded gauge in the concrete deck (Figure 3-A)—or lightly instrumented, typically with four gauges (one per flange tip). The strain gauges were spot-welded to the steel girder and embedded in the concrete (as applicable). A complete description of the heavily and lightly instrumented cross-sections is presented in Volume I (Zhou et al. 2026b).

For cross-sections instrumented with at least one gauge per flange tip, measured strains were decomposed into strong-axis bending, flange lateral bending (comprising weak-axis bending and warping torsion effects), axial force, and local bending at the bottom flange and web (Greimann et al. 2014). Strains were then used to calculate stresses assuming elastic response under service live load. Volume I provides a detailed description of the data-processing scheme (Zhou et al. 2026b).

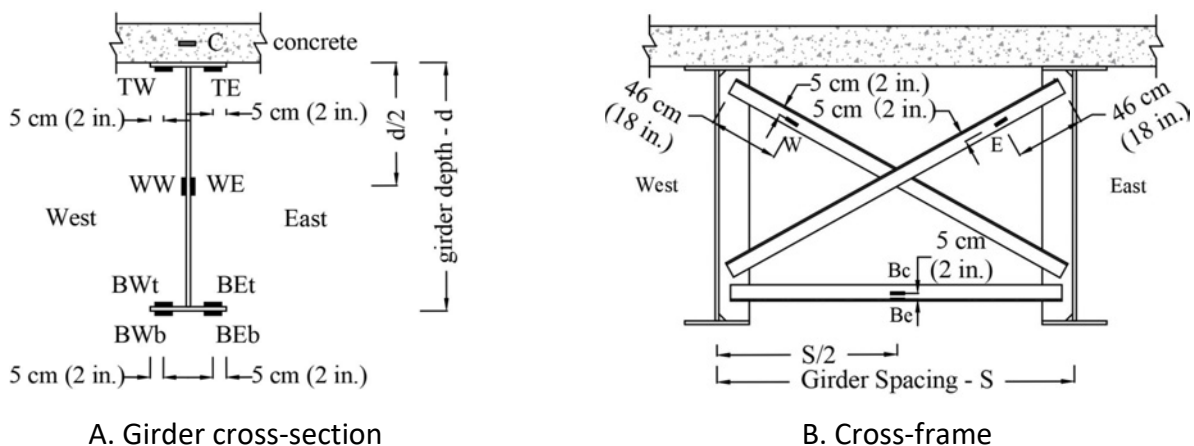


Figure 3. Cross-section view. Mattis-57 and Mattis-74 strain gauge locations.

Several cross-frames near the heavily instrumented girder cross-sections and close to bridge corners were monitored (Figures 1 and 2). As shown in Figure 3-B, three strain gauges were typically installed at an instrumented cross-frame location: at the west top portion of one diagonal angle (W), at the east top portion of the other diagonal angle (E), and at mid-length of the bottom chord angle on the connection (concentric) leg (Bc). For location CF1-1&2 on Mattis-74, an additional strain gauge was placed at mid-length of the bottom chord angle on the outstanding (eccentric) leg, named CF1-1&2-Be.

Tiltmeters (“T” in Figures 1 and 2) were installed at the two ends of G5 for Mattis-74 and G6 of Mattis-57 to measure girder end rotations. Linear displacement transducers (crackmeters, “C”) were installed at the four corners (C-SW, C-NW, C-SE, and C-NE in Figures 1 and 2) of each bridge to measure relative movement between the bridge superstructure and substructure. Crackmeters were installed along and perpendicular to the girder line for the stub abutment bridge (Mattis-74, Figure 1) between the abutment backwall and girder bottom flange (at the bearing level), and only along the bridge longitudinal direction for the IAB (Mattis-57, Figure 2), between the approach slab and transition slab (at the deck level).

CURRENT RESEARCH AND REPORT OUTLINE

Two steel I-girder skewed bridges were monitored in Champaign, Illinois. The instrumentation plans are shown in Figures 1 and 2. The data collected from these bridges has helped to (1) validate models for short-term (deck placement and live load) demands (Volume II) and long-term demands (thermal), and (2) study the field behavior of stub and integral abutment skewed bridges.

This report (Volume III) presents the following items. The updated numerical simulation methods and parametric study matrix used in the studies (Chapter 2), the evaluation of girder weak-axis bending on skewed IABs under live loads through a parametric study of selected bridges built in Illinois (Chapter 3), a summary of the field-collected data since September 2021 through December 2025 (Chapter 4), and the evaluation of skewed bridge response under thermal loads through a parametric study of selected bridges built in Illinois (Chapter 5). Conclusions and recommendations for potential applications are presented in Chapter 6. Additionally, Appendix A presents the complete set of field-collected data for both monitored bridges, while Appendix B presents the extended model validation information. This research project was carried out in the following four tasks, and this report supports the results in Tasks 2 through 4.

- **Task 1: Literature review and agency survey.** A review of the existing research literature and typical design practices for steel I-girder bridges was conducted at the beginning of and throughout the project to inform, guide, and provide context to the current research. A summary of the literature review and the findings from responses to the agency survey are presented in Volume I (Zhou et al. 2026b). Additional discussion of relevant topics is presented in this report as needed.
- **Task 2: Field monitoring.** Informed by findings from the agency survey, two new IDOT steel I-girder bridges were selected for a long-term field-monitoring campaign to collect comprehensive data that document behavior from construction to after bridges are in service.

Two bridges skewed around 45° (one with stub abutments and the other with integral abutments) were monitored in the field. Strain and temperature variation at critical girder cross-sections and cross-frames are monitored, and bridge global movement (girder end rotations and bridge corner displacements) is captured. Volume I presents information for the selected bridges, detailed instrumentation plan, and processing scheme of field measurements (Zhou et al. 2026b). Observations based on the field measurements are presented in Chapter 4. The long-term field-collected data for all installed gauges are presented in Appendix A in both raw stress and decomposed stress formats.

- **Task 3: Numerical simulation.** Numerical simulations are carried out in the current research to enhance understanding of field measurements by modeling the bridges under field monitoring and to provide insights on geometric effects and abutment type on skewed steel I-girder bridge behavior through parametric studies. Three-dimensional finite element analysis methods have been developed since the beginning of the project. Iterations to improve the accuracy of computational modeling have been conducted with field measurements after data collection started. Numerical modeling methods and preliminary numerical simulations are presented in Volume I (Zhou et al. 2026b). Additional finite element model validation and modeling insights for thermal demands are presented in Chapter 2, and observations based on parametric studies are presented in Chapters 3 and 5 of this report.
- **Task 4: Design recommendations.** The prior three tasks are synthesized in Chapter 6. Observations and insights on the behavior of skewed steel I-girder bridges are summarized, and analysis, design, and construction guidelines are proposed to implement the research results in practice.

CHAPTER 2: NUMERICAL SIMULATION AND PARAMETRIC STUDY MATRIX

NUMERICAL SIMULATION APPROACH

The finite element simulation approach for analyses of bridges under deck placement and live loads is described in detail in Volumes I and II (Zhou et al. 2026a, 2026b). The discussion in this report focuses on the numerical simulation approach for models of bridges subjected to long-term (thermal) loads. A summary of general modeling techniques is also presented in this chapter for reference.

To enhance understanding of field response, parametric studies of bridges subjected to both live (Chapter 3) and thermal (Chapter 5) loads were carried out through 3D finite element analysis using ABAQUS (2023). Steel I-girders, concrete deck, steel cross-frames, steel stiffeners, and abutment backwall and pile cap (for IABs) were all explicitly modeled with 3D 4-node reduced integration shell elements (S4R). Steel end-diaphragms in skewed stub abutment bridges were modeled with 3D 2-node linear frame elements (B31). For the live load parametric study, piers were modeled as springs representing the pier stiffness in both the longitudinal and transverse directions. In contrast, piers were explicitly included in the thermal parametric study. Frame-type piers were modeled using 3D 2-node linear frame elements (B31), whereas rigid wall piers were modeled using the same shell-element type described above. All shell elements were meshed with a typical size of 13 cm (5 in.). The bridge superstructures were considered to behave elastically under dead load, traffic loads, and thermal loads, so linear elastic material properties were used. An elastic modulus of $E_s = 200$ GPa (29,000 ksi) and $E_c = 27.6$ GPa (4,000 ksi) was used for steel and concrete, respectively. Thermal expansion coefficients for steel (α_s) and concrete (α_c) were defined as $6.5 \times 10^{-6}/^{\circ}F$ and $5.5 \times 10^{-6}/^{\circ}F$, respectively.

Girders were defined as continuous over their full length, with tie-constraints for all degrees of freedom (dofs) at girder splices. Connecting plates and stiffeners were modeled as simple rectangular plates connected to girder webs and flanges with tie-constraints in all dofs along all contact edges. Connections between cross-frame members and between cross-frames and transverse stiffeners were simplified as tie-constraints in all dofs. Cross-frame angle end connections were defined to the connected leg of the angle, accounting for eccentric loading with respect to the outstanding angle leg. Haunches were modeled to represent the concrete slab for in-service bridges, and composite behavior was defined with tie-constraints connecting nodes of the haunches and steel girder top flanges in all six dofs.

Boundary conditions at girder ends of stub abutment bridges, and at piers of both stub and integral abutment bridges, were modeled following details from the structural plans of each studied bridge, explicitly accounting for fixed, elastomeric, and high-load multi-rotational (HLMR/pot) bearings. Type I elastomeric bearings, as defined in the IDOT (2023) *Bridge Manual*, were represented using a bilinear frictional behavior, with stiffness characterized in accordance with AASHTO (2024) guidance. A friction coefficient (μ) of 0.45—based on elastomeric bearing experimental behavior (Steelman 2013)—was used to define the force cap, after which no further resistance was assumed.

Fixed bearings were not idealized as perfectly pinned supports. Instead, they were represented using realistic fixed bearing stiffness (Filipov 2012). The fixed-bearing connector response was modeled using a bilinear idealization informed by computational and experimental data summarized by Filipov (2012). The initial stiffness was defined through the assumed anchor bolt yield force (P_y) and yield displacement (Δ_y). The yield force was defined as the sum of the nominal yield capacity in pure shear for all bolts combined, as $P_y = n_b(0.6)F_yA_{be}$, where n_b is the number of anchor bolts, F_y is the anchor bolt yield stress, and A_{be} is the anchor bolt effective area defined by reducing the nominal area (A_b) due to bolt threads as $A_{be} = 0.8A_b$. The yield displacement was defined as $\Delta_y = 0.1(d_b)$, where d_b is the anchor bolt diameter (Filipov 2012). The post-yield behavior was defined consistently using $P_u = n_b(0.6)F_uA_{be}$ and $\Delta_u = d_b$ (Filipov 2012). This modeling choice was particularly important for bridges with rigid walls, where restraint assumptions strongly affected thermal load redistribution and local demands, a phenomenon partially acknowledged in Section 3.8.3 of IDOT's *Bridge Manual* (2023). Additional discussion on flexibility sources of fixed bearing behavior is provided by Filipov (2012) and Steelman et al. (2014). HLMR bearings were modeled as longitudinal rollers while maintaining the same transverse restraint behavior used for fixed bearings.

The bearing behavior was implemented in ABAQUS using connector elements. Support connectors were attached directly between a girder reference point and the explicitly modeled pier structure, rather than to an idealized fixed-reference support. Explicit representation of the pier was critical to capture pier thermal expansion and contraction and to avoid over-constraining the system. A sensitivity study showed that excessive restraint leads to unrealistic lateral bending stresses in bottom flanges.

For IABs, rigid connections between the abutment and superstructure (deck and girders) were represented with tie constraints in all dofs. Backfill soil properties were developed using classical earth pressure theory (with only passive pressure considered) for representative backfill soil material used in Illinois—assumed as loose sand ($\gamma = 125$ pcf, $\phi = 30^\circ$), consistent with previous studies (LaFave et al. 2016b). Elastic perfectly plastic springs were defined at the backfill pressure resultant location (two-thirds of the abutment depth), with maximum passive pressure achieved at 2.5 cm (1 in.) deformation (Clough and Duncan 1991; Shamsabadi et al. 2006). This representation was validated against a more refined representation of the backfill soil over the entire height of the abutment (Olson et al. 2009). P - y curves (Reese et al. 1975; Welch 1972) corresponding to material observed in boring logs for the monitored IAB—identified as stiff clay without free water—were used to develop 6-dof lumped springs, similar to those used in Luo et al. (2017) and implemented under the abutment at each pile location. The simplified 6-dof modeling (Figure 4-A) was validated against a more robust version of the model, where piles and soil surrounding the piles were explicitly included (Figure 4-C). Further details regarding the definition of soil springs for the integral abutments are documented in Volume I (Zhou et al. 2026b).

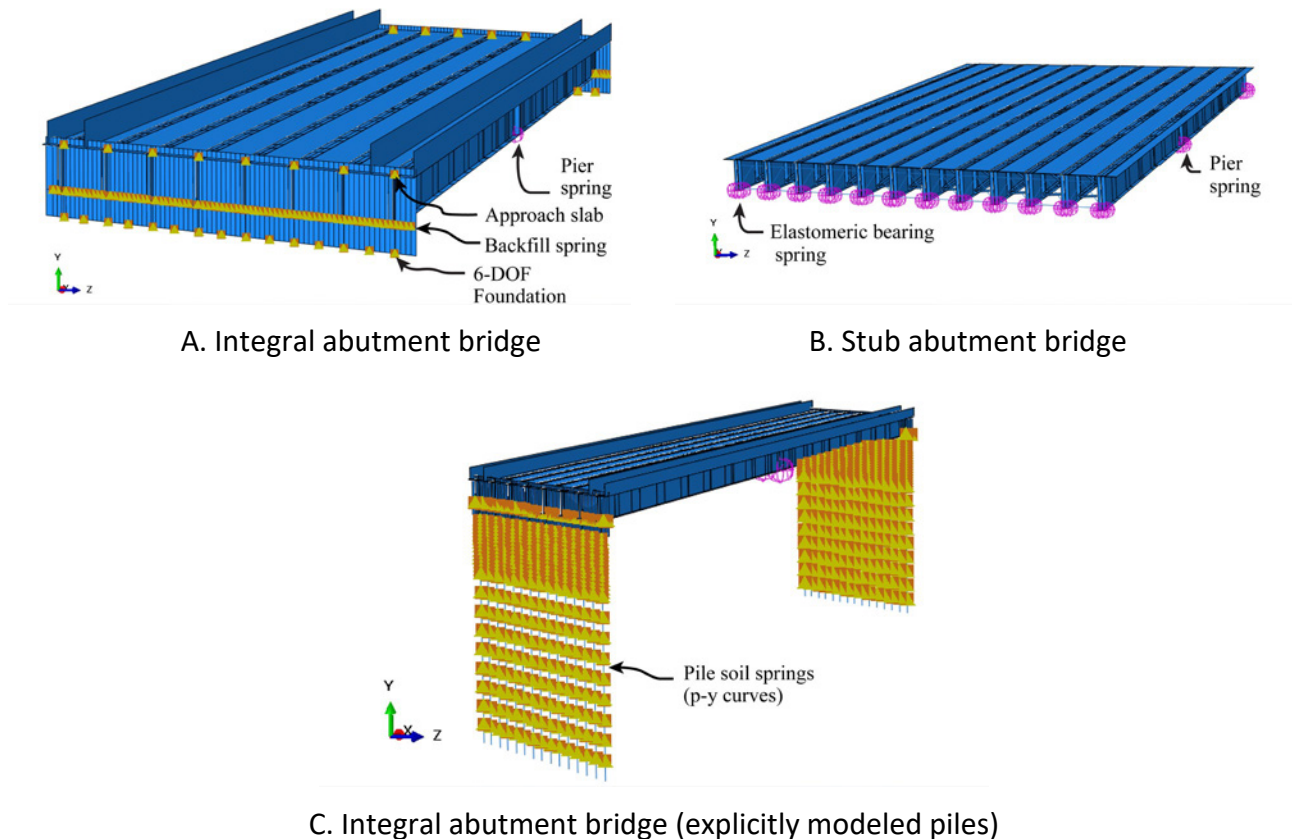


Figure 4. Illustration. Typical 3D finite element models.

Additional sensitivity studies were carried out to evaluate how key modeling parameters influence the response of the bridge under thermal loading. Among the parameters considered—including backfill stiffness, approach slab stiffness, girder-abutment connection stiffness, and the inclusion of wingwalls—the foundation stiffness at the pile level was found to have the most significant impact on the global response. In particular, reducing the lateral stiffness (both parallel and perpendicular to the bridge) of the foundation led to a noticeable decrease in girder moments near the abutments in integral abutment bridges, resulting in improved agreement between the finite element (FE) model and field-measured stresses. This observation highlights the critical role of soil-structure interaction in accurately capturing thermally induced behavior.

To better represent this behavior, cyclic p - y curves were implemented to account for the repeated nature of thermal loading, which induces expansion and contraction cycles in the superstructure. Unlike monotonic p - y relationships, cyclic formulations account for degradation of soil stiffness and resistance with continued loading, as well as the formation of gaps near the ground surface. Following the framework proposed by Welch (1972) and Reese (1975), the cyclic p - y curves were used to reduce the effective soil resistance at the pile level, thereby capturing the softening effects associated with repeated thermal cycles. This approach provides a more realistic representation of long-term foundation behavior in integral abutment bridges, where daily and seasonal temperature

variations can lead to cumulative soil degradation and reduced lateral restraint, ultimately influencing the distribution of forces and moments in the superstructure.

Additional explorations on the accuracy of the IAB model were carried out for the studies presented in this report. First, the model was refined through the inclusion of approach slabs, modeled as linear springs of stiffness equal to the shear stiffness of the fabric bearing pad at the far end approach slab support (near the approach-transition slab intersection). A friction force cap was calculated using a friction coefficient (μ) of 0.45 and a normal force equal to the tributary approach slab weight resting on the bearing pads. Explicitly modeling the approach slab improved the prediction of girder strong-axis bending (under live load) behavior in the vicinity of the abutments. The additional stiffness introduced through the linear spring representation of the approach slab provided more realistic simulation of boundary conditions at the bridge ends, resulting in better agreement with measured strain response. Girder lateral bending response under live load was also slightly improved by the inclusion of approach slabs. Under thermal demands, the inclusion of the approach slab did not have a significant effect, since sliding of the approach slab bearing occurs under small temperature changes.

An additional refinement involved the location of the abutment shell. Modeling the shell at the abutment face and offsetting its properties to the abutment centerline improved the prediction of local stresses. Both the magnitude and behavior of lateral bending at instrumented sections near the abutment were considerably improved. This configuration more accurately captured the distance between the face of the abutment backwall and the first cross-frame, allowing for closer agreement with local measured behavior. A mesh convergence study was conducted for girder shell elements near the integral abutment region to verify stability of stress predictions at the girder-backwall interface, where shell elements are connected through tie constraints. The original mesh size (of approximately 13 cm [5 in.]) was progressively reduced, while stresses from girder elements used in post-processing were monitored. Convergent behavior was observed, indicating mesh-independent stress predictions in the vicinity of the abutment.

Updated IAB Model Validation under Live Loads

Updated model response under live load is compared to measured girder stresses under the truck load test described in Volume II (Zhou et al. 2026a), for representative cross-sections near the abutment and pier, as well as at midspan of the monitored IAB, as presented in Figure 5. Results are presented in terms of decomposed stress components, following the decomposition procedure outlined in Volume I (Zhou et al. 2026b). In this approach, strains measured at the four instrumented flange tips are used to solve for the axial, strong-axis bending, and weak-axis bending components. This allows for direct comparison between measured and simulated strong-axis and weak-axis bending contributions, with weak-axis bending comprising both flange lateral bending and warping torsion effects. The truck loading procedure for the live load validation is described in Volume I (Zhou et al. 2026b).

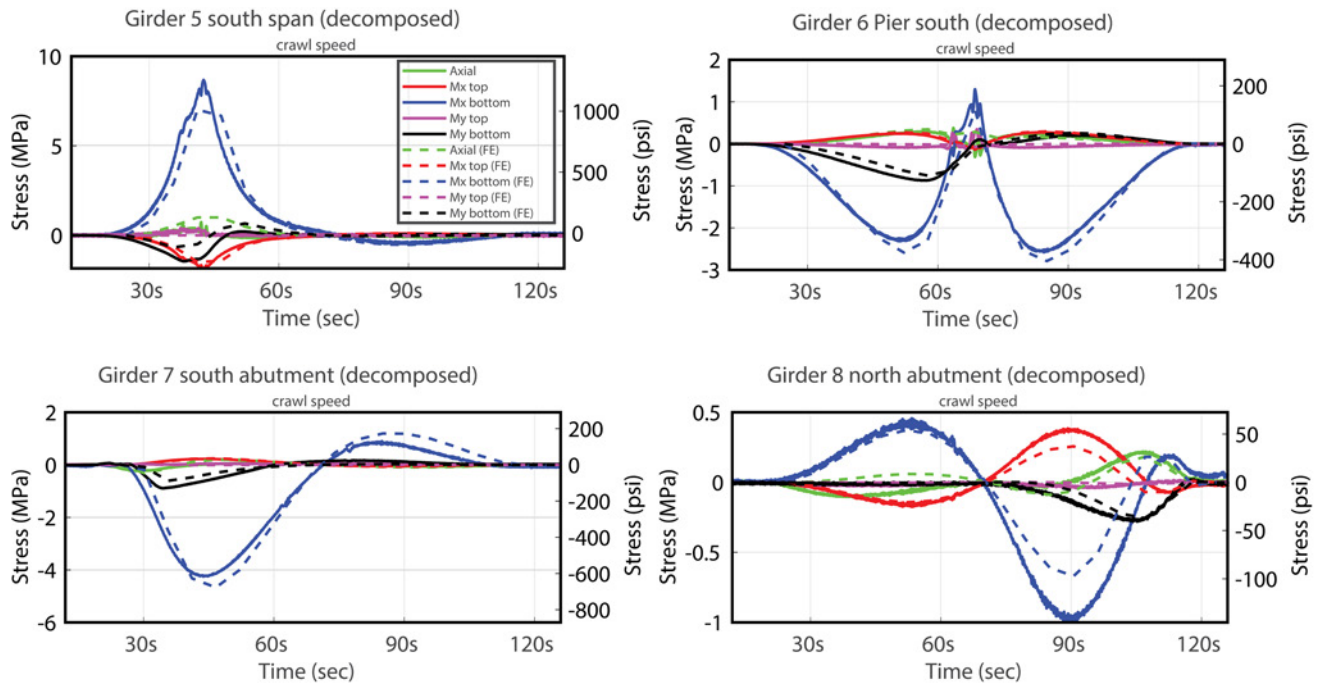


Figure 5. Graph. Updated IAB model validation under live loads.

IAB Model Validation under Thermal Loads

Validation of the numerical models under thermal loading was performed by directly subjecting the ABAQUS models to temperature variations measured in the field, with an objective of reproducing the long-term thermally induced behavior of the bridges. Particular emphasis was placed on capturing seasonal trends, which—in most cases—govern the dominant stress variations in integral abutment bridges, rather than short-term daily fluctuations. While daily temperature cycles can also induce meaningful stress fluctuations, their effects are influenced by complex thermal gradients—both vertically through the depth of the superstructure and horizontally across its width—that are not explicitly simulated in the present study. The magnitude of these daily thermally induced stress variations is further evaluated in the following sections using field-measured stress fluctuations. To focus on the dominant long-term (seasonal) thermal effects, a moving mean was computed from the measured mean temperature (all available thermistors) data over the monitoring period spanning September 2021 through December 2025. This smoothed mean temperature history was subsequently downsampled to reduce computational demands while preserving long-term trends (see Figure 6). The resulting temperature input was then applied uniformly to the FE model. In the figure legend, the number of data points used for each data set is denoted by “n.”

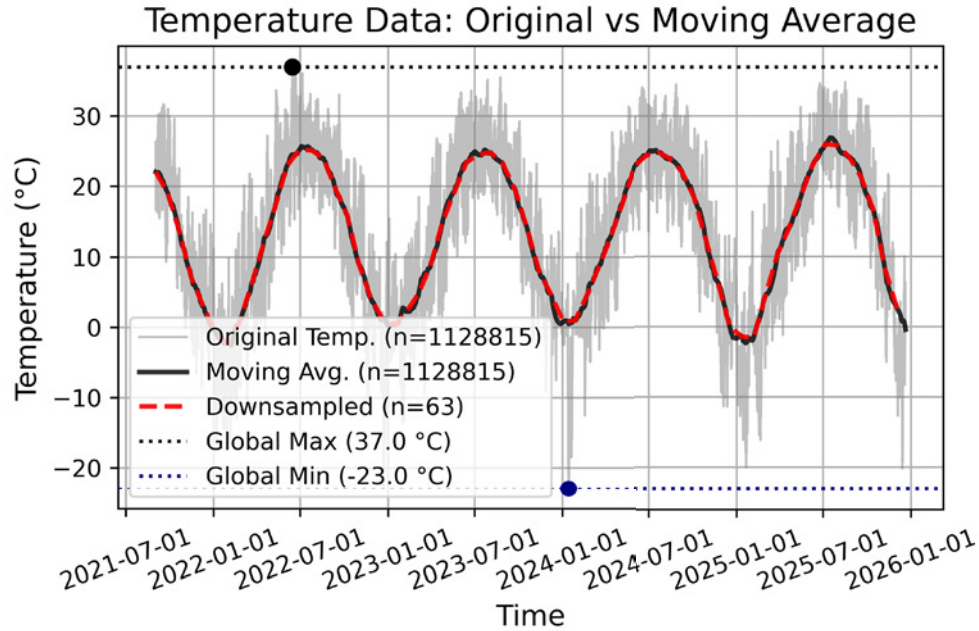


Figure 6. Graph. Temperature variations for FE model thermal validation.

The primary validation of the model is conducted through temperature-stress relationships, presented in Figures 7 through 9. These plots directly capture the dominant (long-term) thermally induced behavior of the bridge and provide a clear basis for comparison between FE-predicted and field-measured responses. The slope of these plots represents the change in stress per unit change in temperature at each location. Therefore, agreement between the FE-predicted and field-measured slopes indicates that the model is adequately capturing the long-term thermally induced behavior of the bridge. Deviations from this slope, on the other hand, may reflect short-term daily fluctuations with a greater influence of thermal gradients, an effect discussed in Chapter 4. The validation presented herein consists of data from bottom flange gauges at representative cross-sections of the monitored integral abutment bridge, including locations near the abutments, at the piers, and at midspan, capturing variations in structural response across different regions. Since IABs are more affected by thermal demands due to continuity of the superstructure with the substructure (and resulting greater restraint), this section focuses on IAB model validation.

In addition to the temperature-stress comparison, response history comparison for selected gauges is shown in Figure 10 to illustrate the temporal consistency of the response for selected cross-sections. In these figures, the raw field temperature, the processed temperature input used in the FE model (i.e., the downsampled moving mean), the FE-extracted stresses, and the measured field stresses are presented concurrently. To ensure consistency in the comparison, a moving mean of the field-measured stresses—computed using the same window employed for the temperature processing—was also included in the response history plots, enabling a direct evaluation of long-term trends between the numerical and field response. Additional validation results for all instrumented gauges are provided in Appendix B, where the full set of temperature-stress comparisons is included for completeness.

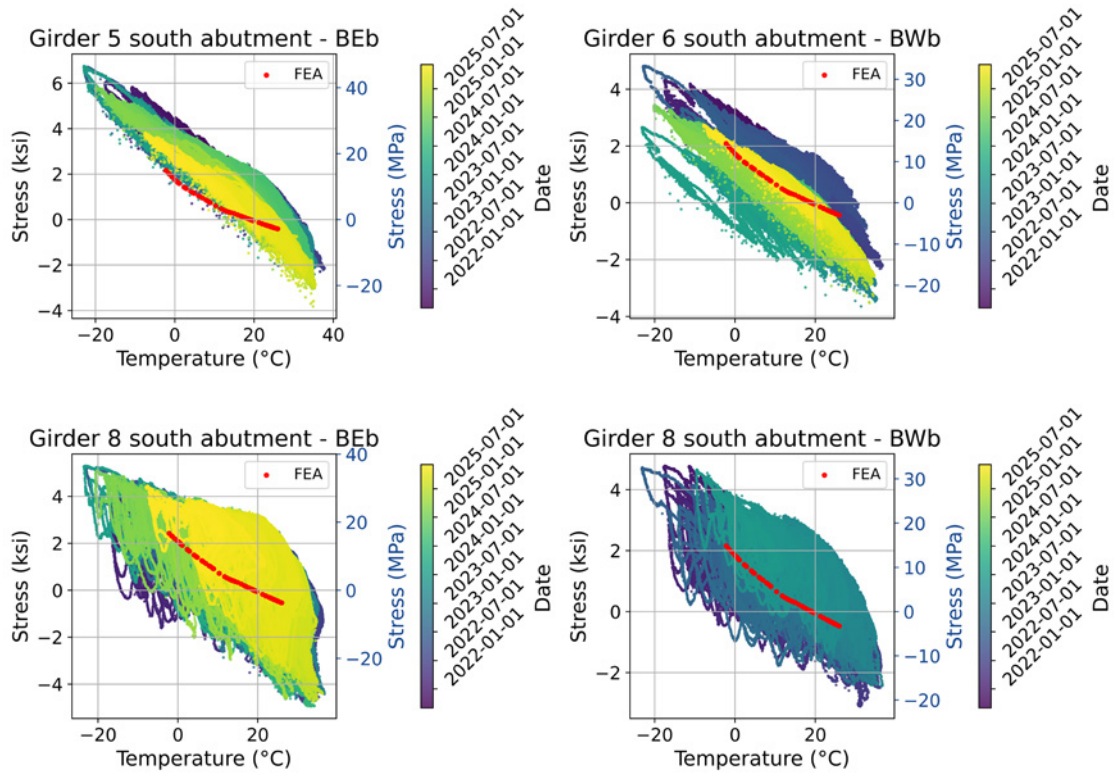


Figure 7. Graph. Temperature-stress comparison (FE vs. field data) near abutment regions.

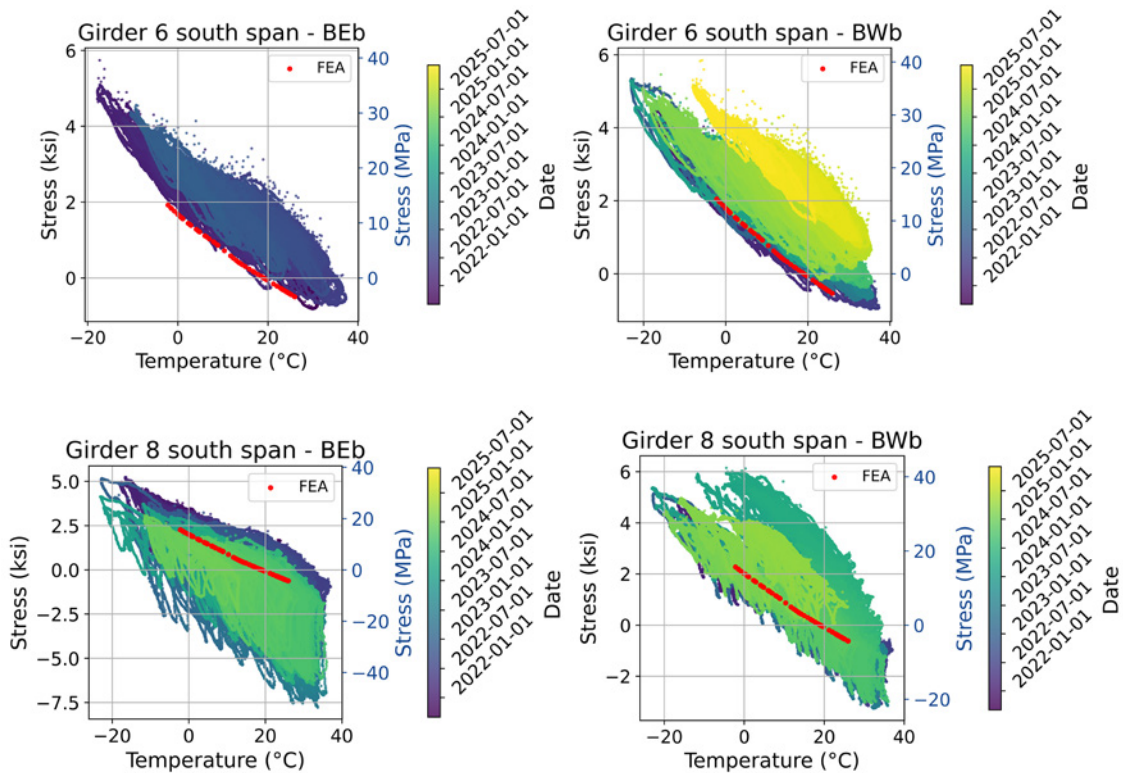


Figure 8. Graph. Temperature-stress comparison (FE vs. field data) near span regions.

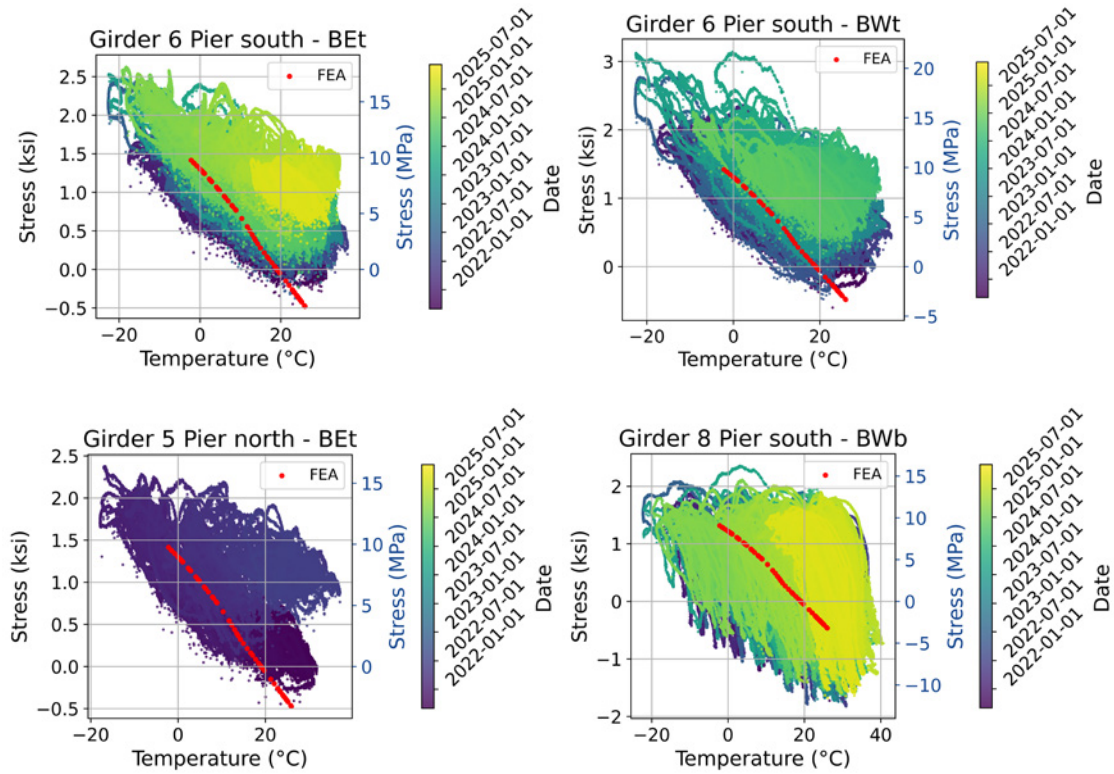
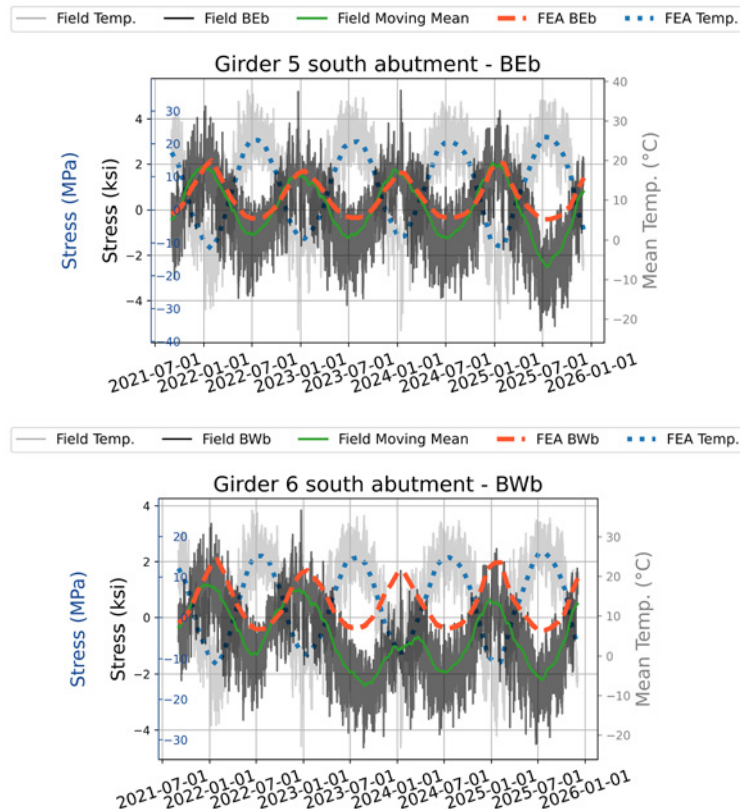


Figure 9. Graph. Temperature-stress comparison (FE vs. field data) near pier regions.



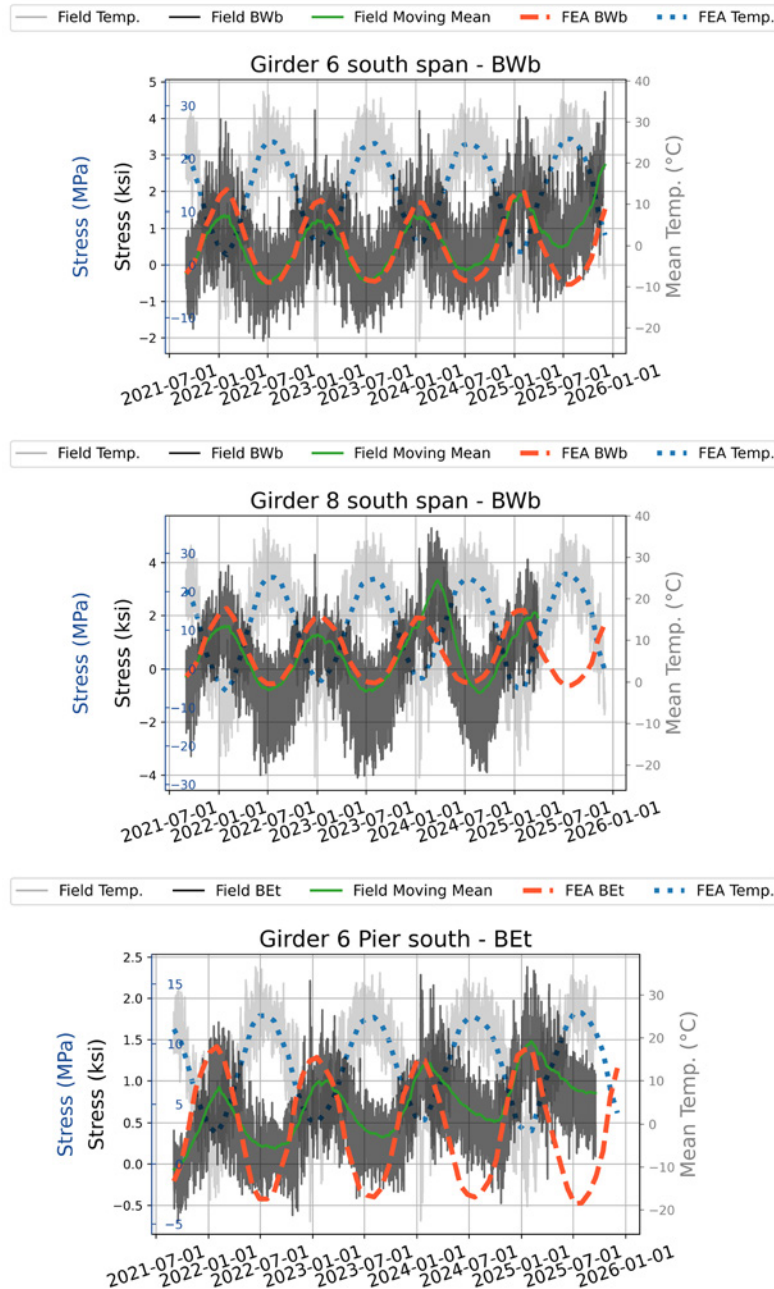


Figure 10. Graph. Response history stress comparison (FE vs. field data) at different cross-sections.

The response history comparisons in Figure 10 demonstrate that the FE-predicted response generally follows the long-term field trend represented by the moving mean, indicating that the model adequately captures the dominant seasonal thermally induced behavior of the bridge. Nevertheless, some deviations from the typical long-term behavior are observed in the measured response throughout the monitoring period, suggesting that the structural response of the integral abutment bridge may evolve over repeated thermal cycles.

Complementary to the total stress comparisons, Figure 11 and Figure 12 present temperature-decomposed stress relationships at selected representative locations. By separating the stress response into its contributing components (e.g., strong-axis bending, weak-axis [lateral] bending, axial), these plots provide additional insight into the mechanisms governing the thermally induced behavior. The data presented herein focus on axial stresses; some bottom-flange lateral bending stress comparison plots are also shown. Appendix B presents the complete validation set for decomposed stresses.

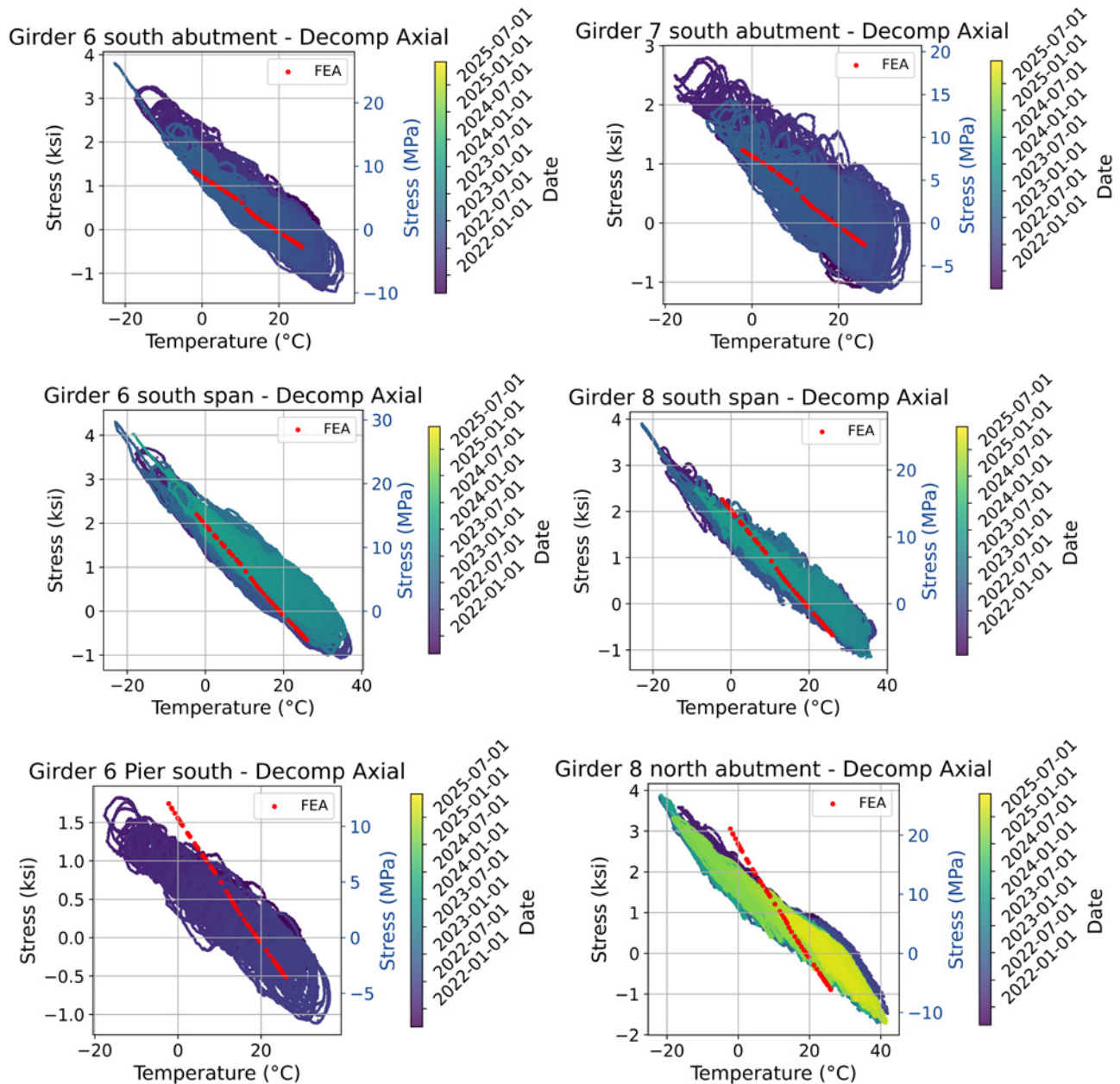


Figure 11. Graph. Temperature-axial stress comparison (FE vs. field data) at different cross-sections.

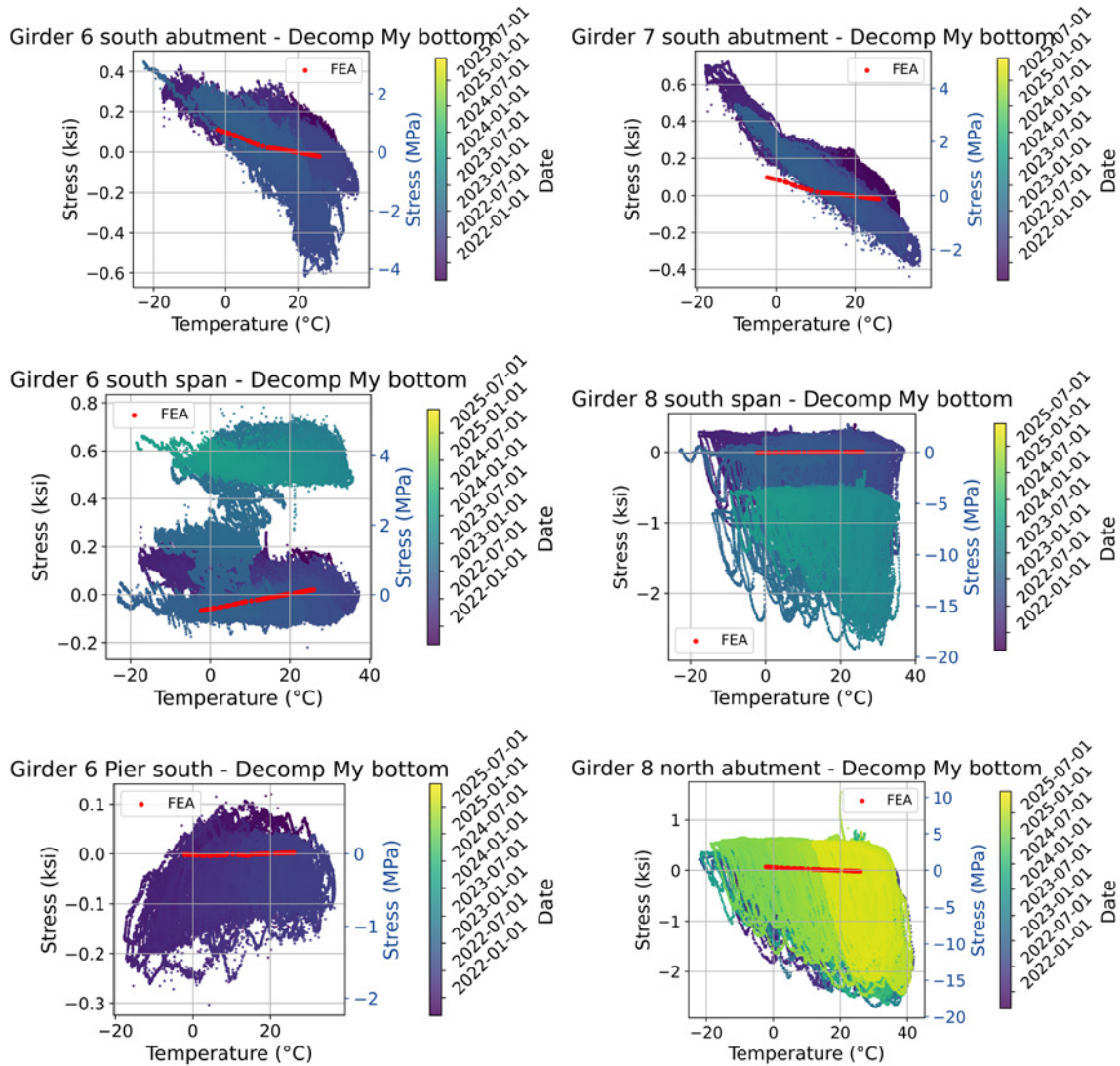


Figure 12. Graph. Temperature-bottom flange lateral bending stress comparison (FE vs. field data) at different cross-sections.

Excellent agreement between the FE model and field measurements was observed for the axial stress component at all monitored cross-sections, indicating that the numerical model can adequately capture the dominant long-term thermally induced axial behavior of the bridge. In particular, the model consistently reproduced the seasonal stress trends and overall magnitude of the measured axial response, providing confidence in its capability to predict global thermally induced axial stresses throughout the structure.

For the lateral bending component, larger short-term fluctuations and deviations between the numerical and measured responses can be observed. These differences suggest that thermal gradients have a more significant influence on lateral bending behavior than on the axial response, and such effects were not explicitly simulated in the present study. The magnitude and characteristics of these deviations from the long-term trends are further discussed in Chapter 4.

Figure 13 presents selected comparisons between FE-predicted and field-measured cross-frame responses as a function of temperature. These results are used to evaluate the ability of the model at reproducing thermally induced force and stress demands in the cross-frame system, which is critical for load redistribution between girders.

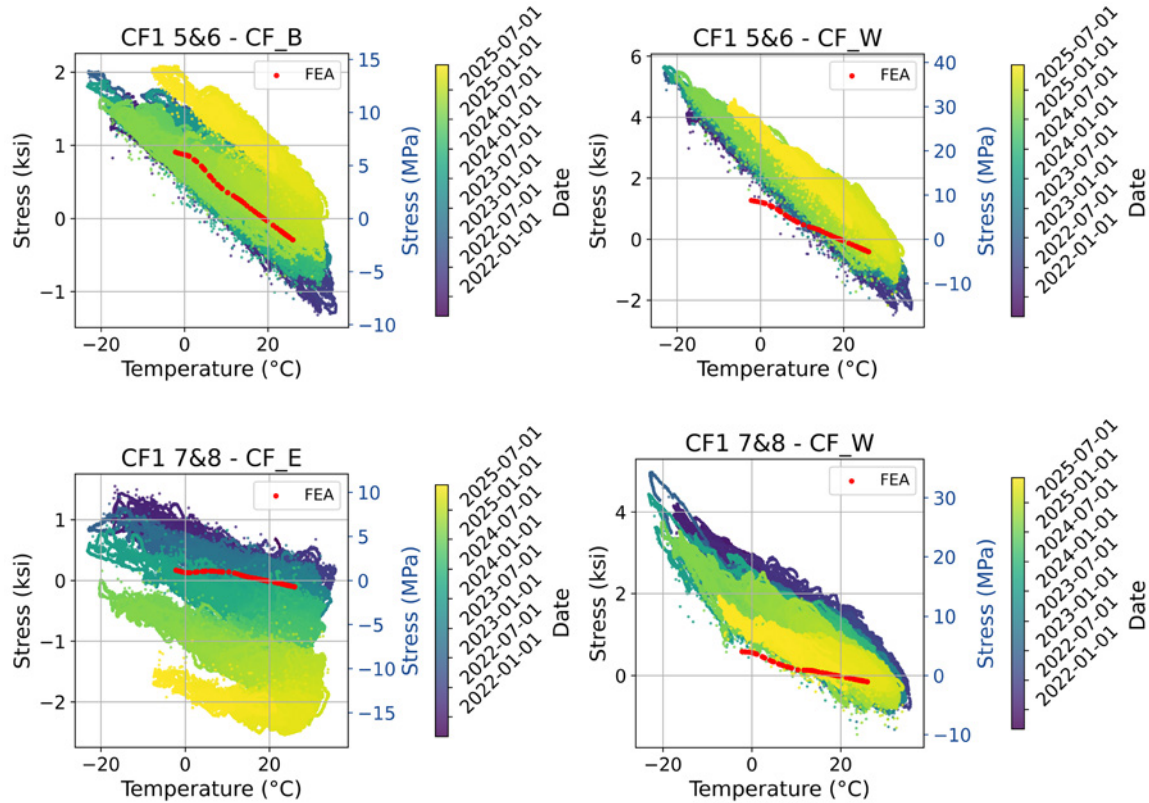


Figure 13. Graph. Temperature-stress comparison (FE vs. field data) at different cross-frames.

Overall, the FE model was able to reproduce the dominant long-term temperature-stress behavior observed in most of the monitored cross-frame gauges, with generally good agreement in both the slope sign and overall trend of the temperature versus stress response. Capturing this relationship provides significant confidence that the numerical model is adequately representing the thermal behavior governing the cross-frame system, even when these forces can be strongly influenced by the complex in-plane rotations of the superstructure associated with skewed bridges. Although some individual gauges exhibited underestimation of thermally induced stress magnitudes and localized deviations from the typical long-term behavior throughout the monitoring period, the FE model consistently captured the dominant long-term thermal response characteristics of most monitored cross-frames.

Figure 14 presents comparisons between FE-predicted and field-measured crackmeter responses for all available instrumentation locations. These measurements capture relative displacements at key locations of the structure and provide a direct assessment of global bridge movements under thermal loading.

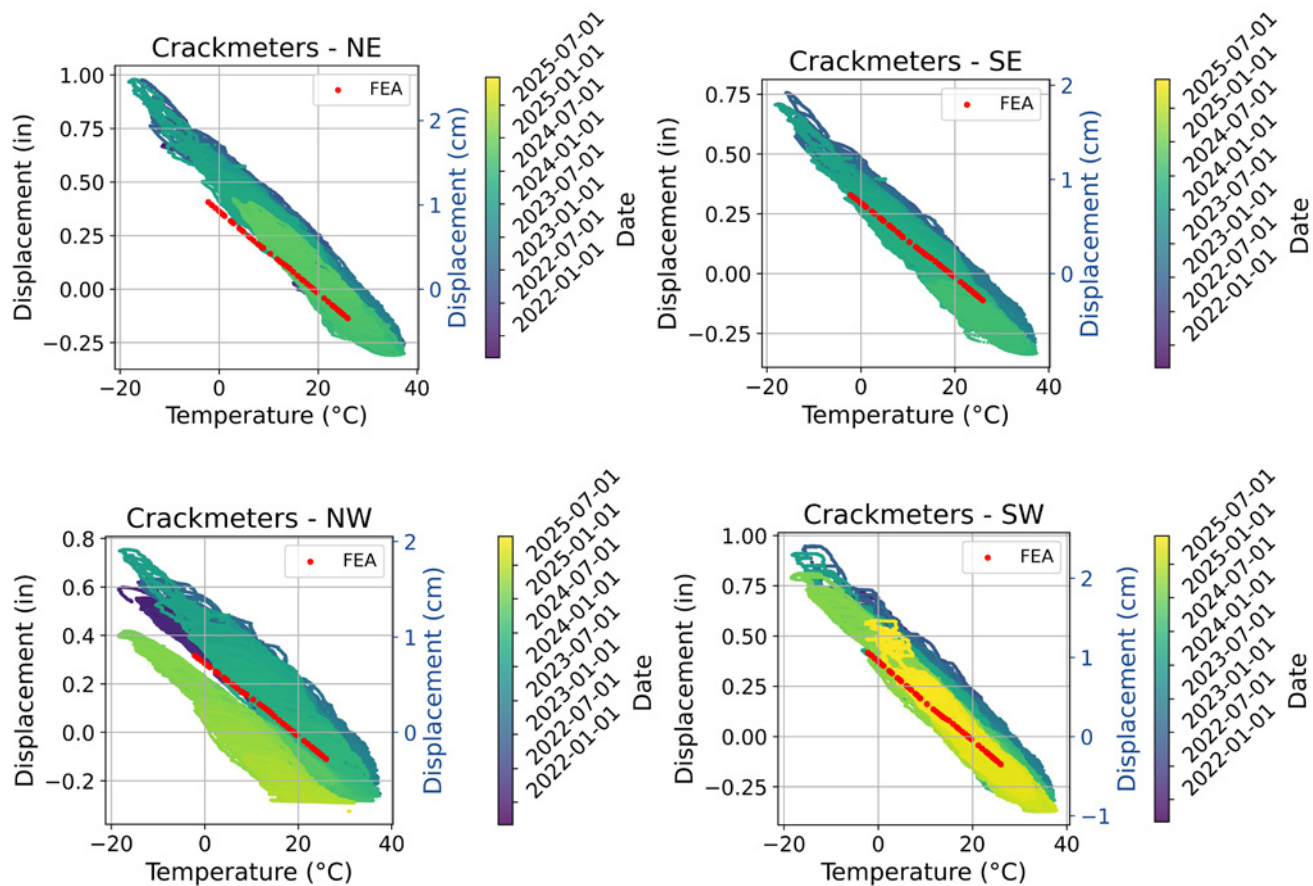


Figure 14. Graph. Temperature-displacement comparison (FE vs. field data) at all four corners of the monitored IAB.

Overall, excellent agreement was observed between the FE-predicted and field-measured responses for all four crackmeters, with the numerical model accurately reproducing both the magnitude and long-term trends of the measured thermally induced displacements. This level of agreement provides strong confidence that the longitudinal restraint behavior of the bridge is being adequately captured and indicates that the boundary conditions implemented in the numerical model appropriately represent the actual structural response under thermal loading. This good agreement was achieved despite the field crackmeter displacements being measured at the approach-transition slab, while the FE displacements were extracted at the exterior girder top flanges where they connect to the abutment backwall.

Figure 15 presents comparisons between FE-predicted and field-measured tiltmeter responses for both bridge ends. The rotational behavior is more complex than other bridge responses and appears to vary throughout the temperature range. This behavior is further discussed in Chapter 4 through a more thorough analysis of the field-measured response.

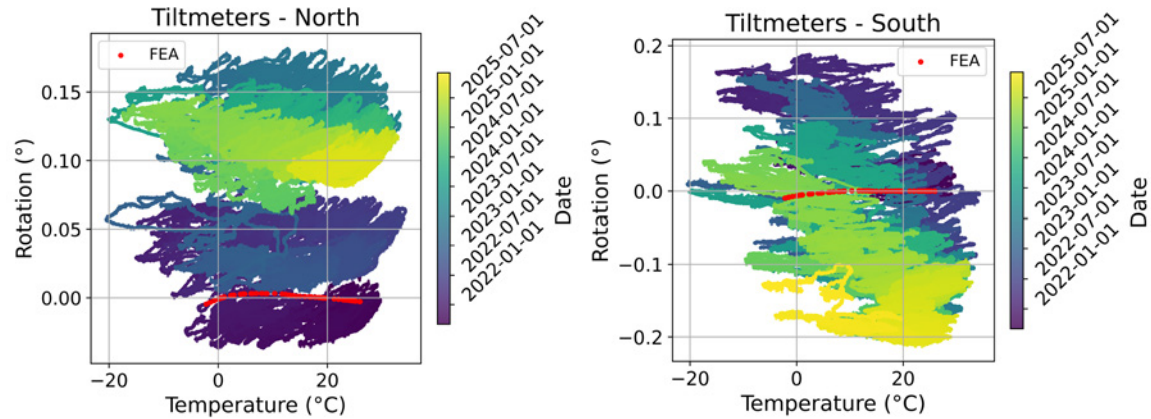


Figure 15. Graph. Temperature-rotation comparison (FE vs. field data) at bridge ends.

PARAMETRIC VARIATIONS

Parametric variations were selected for study from an assessment of a bridge database provided by IDOT, which includes all steel bridges built in Illinois between 2015 and 2020. Bridges believed to provide the most meaningful insights on spatial and temporal load distribution and sources of stresses in skewed and integral abutment I-girder bridges were selected for study. By using bridges with elements designed per the current IDOT manual (IDOT 2023) and AASHTO LRFD specifications (AASHTO 2024), the efficiency or conservatism of such standardized design approaches can be assessed.

Table 1 presents the 20 bridges selected and the parameter variations considered in this study. Bold font indicates properties of the existing bridges, while italics denote the field-monitored bridges. The remaining entries correspond to bridge variations analyzed parametrically. Abbreviations used in the table are defined as follows: Ch-Cont denotes channel diaphragms in a continuous configuration; Ch-Stag denotes channel diaphragms in a staggered configuration (perpendicular to girders); Ch-Par denotes channel diaphragms in a continuous configuration, parallel to skew; X-Cont denotes X-type cross-frames arranged in a continuous configuration; and X-Stag denotes X-type cross-frames arranged in a staggered configuration (perpendicular to girders). The parameter D/b_f refers to the ratio of girder depth (D) to flange width (b_f). Full structural plans for each bridge are publicly available through an IDOT ePlan internet search by contract number. Note that if an original bridge had 0° skew and cross-frames arranged in a continuous configuration, then the skewed variations were modeled with a staggered configuration, having a cross-frame longitudinal offset equal to the offset of the girders due to skew.

Table 1. Parametric Study Matrix

Number of Span	IDOT Contract Number	Bridge Length m (ft)	Deck Width m (ft)	Skew Angle	Abut. Type	Cross-frame Type	Min. D/b_f	Max. D/b_f
Single-Span	74106	13.7 (45)	9.1 (30)	0° , 15°, 30°, 45°, 60°, 70°	IAB , Stub	Ch-Cont	2.64	2.64
	74223	21.3 (70)	10.7 (35)	0°, 15°, 35° , 45°, 60°, 70°	IAB , Stub	Ch-Stag	2.88	2.88
	66994	27.4 (90)	9.1 (30)	0°, 15°, 30°, 45° , 60°, 70°	IAB , Stub	Ch-Stag	3.20	3.20
	70534	39.6 (130)	10.7 (35)	0°, 15°, 30° , 45°, 60°, 70°	IAB , Stub	X-Stag	3.11	3.11
2-Span Continuous	66F57	33.5 (110)	12.2 (40)	0°, 15°, 30°, 50° , 60°, 70°	Stub , IAB	Ch-Stag	2.92	2.92
	74323	45.7 (150)	9.1 (30)	0°, 15°, 30°, 45° , 60°, 70°	IAB , Stub	Ch-Stag	2.15	2.15
	70A61	76.2 (250)	15.2 (50)	0°, 20° , 30°, 45°, 60°, 70°	IAB , Stub	Ch-Stag	2.78	3.30
	<u>70B38 (I-57)</u>	<u>106.7 (350)</u>	<u>15.2 (50)</u>	0°, 15°, 30°, <u>45°</u> , 60°, 70°	<u>IAB</u> , Stub	<u>X-Stag</u>	<u>3.80</u>	<u>4.21</u>
	<u>70B38 (I-74)</u>	<u>94.5 (310)</u>	<u>22.9 (75)</u>	0°, 15°, 30°, <u>41°</u> , 60°, 70°	<u>Stub</u> , IAB	<u>X-Stag</u>	<u>3.34</u>	<u>4.48</u>
3-Span Continuous	78522	36.6 (120)	12.2 (40)	0°, 15°, 30° , 45°, 60°, 70°	IAB , Stub	Ch-Stag	2.67	2.67
	78263	39.6 (130)	9.1 (30)	0°, 15°, 30°, 45° , 60°, 70°	IAB , Stub	Ch-Cont	2.67	2.67
	66A57	39.6 (130)	9.1 (30)	0°, 15°, 30° , 45°, 60°, 70°	IAB , Stub	Ch-Stag	2.83	2.83
	60R95	143.3 (470)	22.9 (75)	0°, 15°, 30° , 45°, 60°, 70°	IAB , Stub	X-Stag	3.27	4.45
	62F95	143.3 (470)	21.3 (70)	0°, 15°, 30°, 50° , 60°, 70°	Stub , IAB	X-Stag	2.88	3.40
	74180	158.5 (520)	10.7 (35)	0° , 15°, 30°, 45°, 60°, 70°	IAB , Stub	X-Cont	3.39	3.53
4-Span Continuous	68C57	67.1 (220)	16.8 (55)	0°, 15° , 30°, 45°, 60°, 70°	IAB , Stub	Ch-Par	2.03	2.03
	70545	79.2 (260)	16.8 (55)	0°, 15°, 25° , 45°, 60°, 70°	IAB , Stub	Ch-Stag	1.97	1.97
	68B69	97.5 (320)	10.7 (35)	0°, 15°, 35° , 45°, 60°, 70°	IAB , Stub	Ch-Stag	2.08	2.14
	74324	112.8 (370)	9.1 (30)	0°, 15°, 30°, 45° , 60°, 70°	IAB , Stub	Ch-Stag	2.21	2.21
	68B44	198.1 (650)	10.7 (35)	0° , 15°, 30°, 45°, 60°, 70°	Stub , IAB	X-Cont	3.69	3.79

Note: **Bold** indicates an original bridge parameter, while underlined italics denote the field-monitored bridges.

Additionally, Table 2 summarizes the abutment and pier support configurations of the bridges included in the parametric study. The table helps identify the boundary conditions considered in the study, which are particularly relevant for skewed steel I-girder bridges subjected to thermal loading.

For the parametric variations, cross-frame spacings were adjusted to account for changes in bridge skew while preserving span lengths. With an objective of keeping the distance from the abutment face to first cross-frame constant (equal to that of the original bridge), exterior spacings (between abutment centerlines and first cross-frames) were modified based on the change in beam longitudinal offset and abutment thickness projection induced by skew, with intermediate cross-frame spacings in end spans proportionally adjusted to maintain the span length. Cross-frame spacings between interior piers and from the final cross-frame of each end span to the adjacent pier were kept unchanged. Cross-frame spacings and locations were identical for both abutment variations of the same bridge.

Table 2. Support Type of Bridges Selected for Parametric Study

Number of Span	IDOT Contract Number	Left Abutment	Bearing Type Pier 1	Bearing Type Pier 2	Bearing Type Pier 3	Right Abutment	Pier Type
Single-Span	74106	Integral	–	–	–	Integral	–
	74223	Integral	–	–	–	Integral	–
	66994	Integral	–	–	–	Integral	–
	70534	Integral	–	–	–	Integral	–
2-Span Continuous	66F57	E. type I	Fixed	–	–	E. type I	Rigid wall
	74323	Integral	Fixed	–	–	Integral	Rigid wall
	70A61	Integral	Fixed	–	–	Integral	Frame
	<u>70B38 (I-57)</u>	<u>Integral</u>	<u>Fixed</u>	–	–	<u>Integral</u>	<u>Frame</u>
	<u>70B38 (I-74)</u>	<u>E. type I</u>	<u>Fixed</u>	–	–	<u>E. type I</u>	<u>Frame</u>
3-Span Continuous	78522	Integral	Fixed	Fixed	–	Integral	Rigid wall
	78263	Integral	Fixed	Fixed	–	Integral	Rigid wall
	66A57	Integral	Fixed	Fixed	–	Integral	Frame
	60R95	Integral	Fixed	Fixed	–	Integral	Frame
	62F95	HLMR long.	HLMR long.	HLMR fixed	–	HLMR long.	Frame
	74180	Integral	Fixed	Fixed	–	Integral	Frame
4-Span Continuous	68C57	Integral	E. type I	E. type I	E. type I	Integral	Rigid wall
	70545	Integral	Fixed	Fixed	Fixed	Integral	Rigid wall
	68B69	Integral	Fixed	E. type I	Fixed	Integral	Rigid wall
	74324	Integral	E. type I	Fixed	E. type I	Integral	Rigid wall
	68B44	E. type I	Fixed	Fixed	Fixed	E. type I	Frame

Note: Underlined italics denotes the field-monitored bridges. “E.” denotes elastomeric bearing.

Additionally, for the IAB variation of bridges originally with stub abutments, an abutment thickness of 112 cm (44 in.) and pile cap height of 107 cm (42 in.) were used. Both are within typical values for integral abutments in Illinois (IDOT 2023). Properties of the backfill soil and soil surrounding the piles for these variations were modeled using those of the monitored IAB, which are representative soil properties in Illinois (Zhou et al. 2026b). The approach slab for all variations was modeled using approach slab properties of the monitored IAB, obtained from the structural plans.

For stub abutment variations of bridges originally having integral abutments, end cross-frames or channels parallel to the abutment were implemented following IDOT’s *Bridge Manual* (2023) typical end diaphragm or cross-frame details, which depend on skew angle and girder web depth.

CHAPTER 3: BRIDGE RESPONSE UNDER LIVE LOAD— PARAMETRIC STUDY

The work presented in Volume II (Zhou et al. 2026a) established the foundation for understanding the live load behavior of skewed steel I-girder bridges through field testing and numerical simulation. Field investigations of the two monitored bridges provided direct insight into static and dynamic bridge response under isolated truck loading, including girder strong-axis bending, lateral bending, cross-frame behavior, and dynamic amplification effects. Complementary 3D finite element analyses were then used to reproduce the field behavior and provide a more comprehensive evaluation of the bridge response beyond the discrete instrumented locations available in the field.

Building on these findings, Volume II further expanded the investigation through numerical parametric studies aimed at evaluating the influence of bridge geometry and support conditions on live load-induced response. Variations in skew angle, bridge width, abutment type, and load path location were examined to better understand how these parameters affect load distribution and girder behavior. Particular emphasis was placed on evaluating the adequacy of current design approaches, including AASHTO live load distribution procedures and simplified recommendations for lateral bending stresses in skewed bridges. These studies provided insight into trends in strong-axis and lateral bending response under isolated truck loading.

The work presented in Volume III expands this previous effort by broadening the scope and objectives of the numerical investigations. While the studies in Volume II focused primarily on understanding live load response for a limited set of monitored bridge configurations and loading scenarios, the present volume evaluates a substantially larger group of representative steel I-girder bridges covering a wider range of geometric characteristics, support conditions, and cross-frame configurations. In addition, the current work places greater emphasis on identifying the spatial distribution of lateral bending demands throughout the bridge system, including regions near abutments, piers, and bridge spans, as well as differences between interior and exterior girders. Particular attention is also given to the role of integral abutment restraint, which was not explicitly addressed in conventional skew bridge studies focused primarily on stub abutment systems.

Through this expanded parametric investigation, Volume III seeks to provide a more comprehensive understanding of the mechanisms governing skew-related lateral bending behavior in steel I-girder bridges and to evaluate the applicability of current simplified design assumptions over a broader range of bridge configurations.

BACKGROUND

Skewed bridge supports complicate the behavior of steel I-girder bridges (Coletti et al. 2011; NSBA 2016; Sanchez 2011; White et al. 2015). Some complications include high cross-frame forces, torsional deformations, transverse load paths, bearing rotations, erection fit-up problems, and significant flange lateral bending. The effects of skewed supports on steel I-girder bridges can be explained as the sum of two factors (Sanchez 2011). First, the high end-cross-frame stiffness induces girders to rotate about the end cross-frame axis (aligned with the skewed support), rather than about

their own perpendicular axis. This rotation results in a lateral flange deformation over the girder length upon application of gravity loads and can be viewed as a global effect of skewed supports. The second factor is related to the enforced compatibility of deformations at cross-frame locations, when cross-frames perpendicular to the girders are used—which connect adjacent girders at locations of different lateral and vertical deformation (due to skewed supports). Forces are then developed in the cross-frames, resulting from deformation compatibility (Sanchez 2011), which causes flange lateral bending that needs to be accounted for in design.

Perpendicular cross-frames are enforced by AASHTO (2024) for bridges with skew greater than 20°. Such a practice improves connection constructability and efficiency of cross-frames as bracing members, while allowing for staggering of diaphragms or cross-frames in adjacent bays between girders. (See the staggered cross-frame configuration in Figure 1.) This staggering of cross-frames helps interrupt and reduce the stiffness of the transverse load path in skewed bridges between obtuse corners (Krupicka and Poellot 1993) by forcing load transfer via girder flange lateral bending (AASHTO 2024). Transverse load paths in skewed bridges are generated due to the interaction of girders and cross-frames, which are primarily intended as bracing for stability. Cross-frames in skewed bridges therefore offer alternative load paths for gravity load between obtuse corners—an effect that is accentuated by the presence of cross-frames near obtuse corners. AASHTO attempts to mitigate this by suggesting a minimum offset of the larger of $4b_f$ and $0.4L_b$ from the support to the first cross-frame, where b_f is the largest girder flange width within the unbraced length and L_b is the unbraced length between the first and second intermediate cross-frame. Complying with the limit of $0.4L_b$, however, may be difficult to achieve due to bracing demands (AASHTO 2024).

If refined analysis is not performed for bridges skewed more than 20°, the suggested AASHTO unfactored lateral flange bending stress is 69 MPa (10 ksi) for interior girders and 52 MPa (7.5 ksi) for exterior girders along the entire bridge length, when discontinuous cross-frame lines occur only near supports. If discontinuous cross-frames extend along the entire bridge length (as in Figure 1), 69 MPa (10 ksi) is used along the entire length for interior girders, while exterior girders use 52 MPa (7.5 ksi) near supports and 14 MPa (2 ksi) along the span. According to AASHTO (2024), these values should be proportioned between dead and live loads according to the unfactored strong-axis bending stresses.

This guidance, however, has created some confusion among researchers and designers due to ambiguity in how the lateral flange bending stress values for exterior girders should be applied (White et al. 2012, 2020; Zhou et al. 2026b). One interpretation is to use 52 MPa (7.5 ksi) near supports and 14 MPa (2 ksi) within the span when discontinuous cross-frames extend along the entire bridge, resulting in two different lateral bending values for the same girder. A second interpretation treats the exterior girder globally. If discontinuous cross-frames occur only near supports, then 52 MPa (7.5 ksi) is applied along the full length of the girder. If they occur along the entire bridge length, then 14 MPa (2 ksi) is applied along the full length of the girder, resulting in a single uniform value. Current IDOT design guidance further simplifies treatment of lateral bending stresses by neglecting such effects for bridges with skew angles less than 45°, while adopting a uniform lateral bending stress of 69 MPa (10 ksi) for all girders and bridge locations when skew exceeds 45° (IDOT *Bridge Manual* [2023] Section 3.3.5). Since the maximum permitted skew angle for integral abutment bridges in current IDOT practice is also 45°, lateral bending stresses are effectively not explicitly considered in

the design of skewed steel I-girder IABs under the present provisions. In this chapter, appropriate treatment of exterior girder lateral bending is examined and supported based on results of parametric analyses.

While the effects of skew and transverse load paths—and the adequacy of AASHTO design values—under dead and live loads have been extensively studied for steel I-girder bridges with stub abutments (Morera 2010; Sanchez 2011; White et al. 2012, 2015, 2020; Zhou et al. 2026a), these effects remain largely unexplored in bridges with integral supports. Studies on IABs have been mainly focused on evaluation of substructure response (Civjan et al. 2013; Fennema et al. 2005; Olson et al. 2009, 2013). Some long-term field monitoring efforts on IAB superstructures have also been reported in the literature (Kim and Laman 2010; Pugasap et al. 2009), although these have primarily focused on long-term behavior of prestressed concrete bridges. On the other hand, Greimann et al. (2014) reported on a field-monitoring campaign and limited numerical studies of curved steel I-girder bridges with integral abutments under live and thermal demands. A series of studies focusing on steel I-girder IAB superstructure performance under thermal demands using both field data (LaFave et al. 2017, 2021) and numerical simulations (LaFave et al. 2016a, 2026b) were then carried out in Illinois. More recently, Zhou et al. (2026a) conducted a limited analysis of lateral bending on steel I-girder IABs under deck placement and live loads, which reported considerable lateral bending stresses in IABs near supports, even for non-skewed bridges.

In the current study, extensive analysis of lateral bending demands in skewed IABs is conducted through 3D finite element (FE) models of 20 representative bridges evaluated at skew angles from 0–70°, presented in Chapter 2 (Table 1). The study examines spatial distribution of girder peak bottom-flange lateral bending stress, compares interior and exterior girder response, and evaluates the influence of cross-frame type and spacing, while also assessing the adequacy of AASHTO-suggested design values considering observed demands. The influence of integral construction—especially the high stiffness of the abutment connection, which effectively fixes the bottom flange—on live load–induced lateral bending is explicitly investigated.

LOADING CONDITIONS

Truck (live) loading for the parametric study was applied using a former AASHTO HS-20 design truck (AASHTO 2024), which is now part of the HL-93 load model per AASHTO (2024) Article 3.6.1.2.2, implemented through a DLOAD ABAQUS subroutine, enabling simulation of moving truck loads at different locations across the bridge deck. Truck positions were incremented longitudinally at 1.5 m (5 ft) intervals along the bridge length, such that the total number of load steps depended on the bridge length and was not a fixed number. Transversely, truck paths were defined using two criteria. First, the total bridge width was divided by 3.65 m (12 ft) to determine the number of design lanes in accordance with AASHTO (2024), and one truck path was assigned to each design lane for every bridge. Second, a transverse influence line analysis was conducted using the cross-sectional geometry of each bridge. In this analysis, single- and multi-truck configurations were positioned transversely to identify locations producing maximum reactions at the girder lines. The bridge cross-section was analyzed by modeling the girder lines as vertical supports, such that the vertical reactions at the supports represent the shear load carried by each girder, which allowed for identification of the truck

position producing maximum demands. Additional truck paths were established at these critical transverse locations. A two-truck configuration with 10% reduction of the nominal axle loads, as specified by AASHTO (2024) for evaluation of negative moment over interior supports, was not implemented in this study. All analyses were conducted using single HS-20 design trucks over the selected lanes. Consequently, maximum negative moment demands at interior piers were evaluated within this single-truck loading framework.

Each truck path was analyzed independently in ABAQUS, with the resulting stresses combined during post-processing to simulate multi-lane analysis. Adequate multiple presence factors were added per AASHTO (2024). The total number of individual analyses resulting from combining all paths for all bridge variations of the parametric study matrix, presented in the next section, sums up to 1,332 ABAQUS analyses. For multi-lane analysis, individual load paths were combined under the assumption that trucks may occupy any longitudinal position along the bridge independently of one another. Only stresses corresponding to the same girder, cross-section, and location in the cross-section were combined prior to identifying governing values. For the total stress response, envelopes for all analysis steps (to account for any possible truck position) were first computed independently for each path and flange location, after which the resulting envelope values were combined for different paths through linear superposition on an element-by-element basis. The governing maximum and minimum total stresses were then independently identified from the combined envelopes for interior and exterior girder groups.

An analogous procedure was followed for determining decomposed stresses. Total stresses were first decomposed into axial and bending components for each analysis step (truck location) and for each truck path. Envelopes of the decomposed components were then computed independently for each path and subsequently combined on an element-by-element basis. Governing maximum and minimum values for each decomposed component were finally identified from the combined envelopes, allowing each component to govern at its own critical location. This approach preserves physical consistency by ensuring that stress decomposition precedes enveloping and that path combination is performed only between stresses associated with identical spatial locations (i.e., the same girder and longitudinal location).

When paths were combined, the considered design truck was positioned transversely such that the center of any wheel load is no closer than 0.6 m (2 ft) from the edge of the design lane (AASHTO 2024). For analysis of the data presented in this study, maximum values for exterior and interior girders were analyzed separately, which allows a direct comparison with AASHTO specifications.

Stress results were extracted at the four flange tips of each girder—at both edges of the top and bottom flanges—using through-thickness averages of the shell element stress at each location. For stub abutment bridges, stress data were retrieved along the entire girder length, up to the last girder element at the abutment. For IABs, stresses were extracted only up to the point where a girder intersects the abutment face. For skewed IABs, girders intersect the abutment face progressively. The flange edge located at the acute corner of the girder-abutment intersection meets the abutment first, while the opposite flange edge continues further along the girder before intersecting the abutment. In this study, stress data were retrieved only up to the longitudinal location at which the acute-side

flange element first meets the abutment face. As a result, a portion of the obtuse-side flange physically extending beyond this point was excluded from stress retrieval, for consistency of full stress decomposition.

OVERALL ANALYSIS OF RESULTS

Before examining the results of bottom-flange lateral bending stress, it is beneficial to first consider the distribution of total stress values (resulting from the combined effects of all beam actions, including strong-axis and lateral bending) across the full parametric dataset. This preliminary assessment provides context for the component-level analyses by illustrating the typical magnitude and variability of stress demand as a function of skew angle when all response components are considered collectively (total stress). Across the dataset, stress demands comprise a wide range, with most values falling below a threshold of approximately 69–90 MPa (10–13 ksi), while isolated cases approach 110 MPa (16 ksi). Establishing these global trends helps frame the subsequent discussion of lateral bending behavior within the broader stress response of the bridges.

Figure 16-A presents the total stress values plotted against skew angle for both interior and exterior girders, highlighting substantial scatter across the full range of considered skews. Each data point corresponds to the maximum absolute value of stress calculated for any load path, at any location of the bridge, determined separately for exterior and interior girders; so, two data points per bridge variation are included. Elevated stress demands are observed at both low and high skew angles. Figure 16-B summarizes the same data using stress envelopes and interquartile range (IQR). The upper bound of the stress envelope decreases from approximately 110 MPa (16 ksi) at low skew to about 83 MPa (12 ksi) at higher skew, while IQR remains relatively stable, with typical stress levels generally between about 35 and 60 MPa (5 and 9 ksi). This global perspective indicates that skew angle primarily influences the dispersion of stress demand rather than its central tendency, motivating a more focused evaluation of individual response components—particularly lateral bending—in the following section.

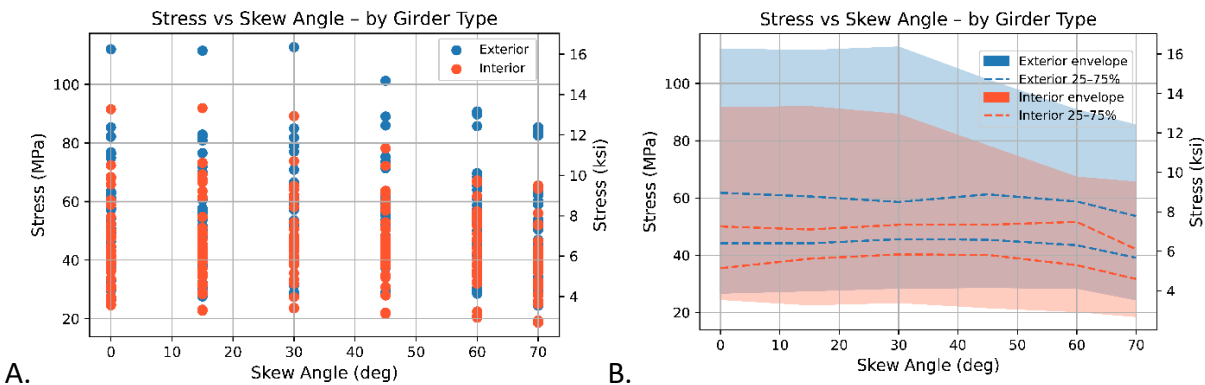


Figure 16. Graph. Total stress vs. skew angle: (A) scatter plot and (B) envelope and IQR.

BOTTOM-FLANGE LATERAL BENDING STRESS

Figure 17 depicts the maximum flange lateral bending stress response anywhere along the bridge. Using this definition, both exterior and interior girders exhibit increasing lateral bending stress magnitudes with increasing skew angle, a behavior that is generally reflected in the lateral bending stress recommendations provided by AASHTO (2024). Interior girder lateral bending stresses increase steadily with skew, with envelope values reaching on the order of 40–50 MPa (6–7 ksi) at higher skew angles and interquartile values approaching approximately 15–28 MPa (2–4 ksi). Exterior girders exhibit lower values, with a maximum of around 21 MPa (3 ksi) at high skew, and an IQR range between 7 and 14 MPa (1 and 2 ksi). For both girder types, the central tendency of response as represented by the IQR begins to stabilize in the range of approximately 50° skew and shows a slight decrease at 70° skew. This behavior suggests that skew-induced amplification of lateral bending stresses may not continue monotonically at moderate to high skew angles.

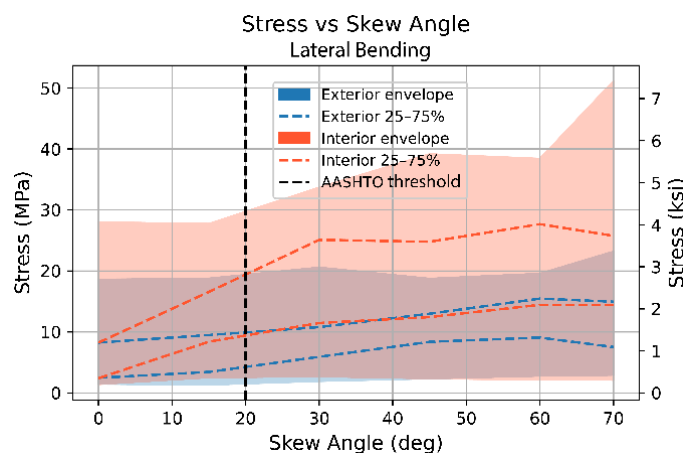


Figure 17. Graph. Envelope and IQR of maximum flange lateral bending stress vs. skew angle.

Before interpreting the flange lateral bending results in the context of code provisions, it is important to establish a consistent basis for comparison between the stresses observed in this study and values recommended by AASHTO. Lateral bending stresses evaluated herein are derived from live load effects only, whereas the AASHTO-recommended limits are expressed in terms of total lateral bending stress, incorporating both dead load (DL) and live load (LL) contributions. To reconcile this difference and enable a more meaningful comparison, guidance provided in AASHTO LRFD Commentary C6.10.1 was adopted, which allows the recommended lateral bending stress limits to be proportioned between dead and live load according to the relative contributions of unfactored major-axis stress at the section under consideration. Accordingly, unfactored DL and LL stresses were extracted from girder moment tables provided on the structural plans for each bridge, which report stresses at pier locations and multiple span sections, resulting in a range of live load proportions across the dataset. While the live load proportion varies between span and pier locations, it is generally larger in span regions. Therefore, the median live load proportion observed at span locations ($= 0.57$) was adopted as a representative value for scaling the AASHTO-recommended total lateral bending stress limits to an equivalent live load component. This approach establishes a

consistent and transparent framework for comparing computed lateral bending stresses to AASHTO recommendations.

In the following sections, flange lateral bending stresses in Figure 17 are compared with the AASHTO lateral bending recommendations on a live load equivalent basis. The AASHTO recommended values are based on a limited examination of refined analysis results for bridges with a range of skews approaching 60° and a depth-to-flange width ratio (D/b_f) of approximately 4.0 (AASHTO 2024), whereas the skew angles and D/b_f ratios of the bridges studied herein range from 0 to 70° and 2 to 4.5, respectively. Additionally, the cross-frame configurations of the studied bridges are mostly staggered, and no lateral bending stress mitigation strategy has been implemented—such as using lean-on bracing (Helwig and Yura 2022; Holt et al. 2022) or omitting selected cross-frames (White et al. 2015)—which allows investigation of upper bound values for lateral bending stress.

For interior girders, the AASHTO-recommended total lateral bending stress limit of 69 MPa (10 ksi) corresponds to an equivalent live load component of approximately 39 MPa (5.7 ksi). Among the set of bridges analyzed in this study, envelope values only exceed such a stress at 70° skew, with the maximum reaching 50 MPa (7.3 ksi) at high skews (as shown in Figure 6). IQR values remain within this limit across the full range of skews. Flange lateral bending stresses are considerable even below the threshold of lateral bending stress applicability (skew of 20°) defined by AASHTO, as well as below the 45° skew threshold adopted by current IDOT design guidance for explicit consideration of lateral bending stresses. It is therefore of interest to explore when such a limit is adequate or inadequate, and if there are specific parameters that define which bridges present significant lateral bending even at low skew and call for consideration of lateral bending stresses in design.

For exterior girders, the AASHTO total lateral bending recommendation—when discontinuous cross-frames are used along the entire length of the bridge—is 52 MPa (7.5 ksi) near supports and 14 MPa (2 ksi) within the span. These values correspond to a scaled live load lateral bending stress of approximately 30 MPa (4.3 ksi) and 8 MPa (1.1 ksi), respectively. The envelope of exterior girder stress is never more than the first limit, but it is greater than the 8 MPa (1.1 ksi) limit across all skew angles. To adequately assess whether the limits are exceeded, though, the data must be evaluated with respect to the location along the bridge where the stresses occur—specifically, whether they develop near supports or within the span. This location-based assessment is presented in the following section.

Bottom-Flange Lateral Bending Stress by Bridge Location

To better understand the distribution of maximum flange lateral bending stresses along the bridge longitudinal axis, Figure 18 separates the maximum lateral bending stress response for exterior and interior girders by the location where the maximum lateral bending stress occurs, distinguishing between abutment, pier, and span regions. Each bridge variation contributes a single data point, with a maximum value at a single location. For purposes of this evaluation, abutment and pier regions are defined as extending 25% of the adjacent span length(s) into the bridge. The remaining portion of each span (the 50% in the middle) is classified as the span region.

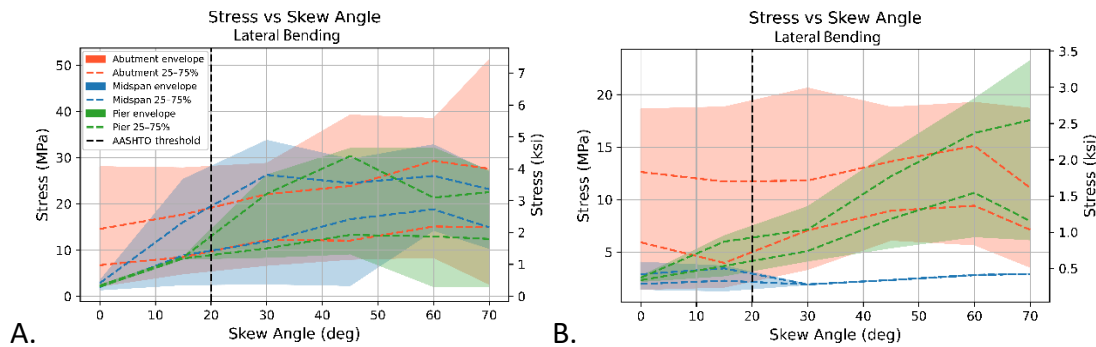


Figure 18. Graph. Envelope and IQR of maximum lateral bending stress vs. skew (by bridge location) for (A) interior girders and (B) exterior girders.

At skew angles below the AASHTO applicability threshold, lateral bending stresses in both interior and exterior girders are governed by responses near the abutment, confirming that high stresses at low skew angles observed in Figure 18 correspond almost entirely to cross-sections located within 25% of the end-span length from an abutment, reaching up to 28 MPa (4.1 ksi) on interior girders and almost 21 MPa (3 ksi) on exterior girders. These observations are also significant in the context of current IDOT practice, since such stress magnitudes develop well below the 45° skew threshold at which lateral bending stresses begin to be explicitly considered by the IDOT *Bridge Manual*. At higher skew angles (> 20°), abutment-region stresses increase for interior girders, peaking at approximately 50 MPa (7 ksi) for very high skew angles, while exterior girder stresses remain relatively constant at about 21 MPa (3 ksi) across skew angles. Similar to observations for low skew, the overall largest lateral bending stresses at higher skew angles generally correspond to cross-sections near the abutments, except at 70° for exterior girders where pier stresses govern.

Stresses near the pier remain low until about 30° skew, after which they increase in both interior and exterior girders. This is even more noticeable in exterior girders relative to their typical values: Peak lateral bending stresses are approximately 30 MPa (4.5 ksi) for 45° skew in interior girders and 23.5 MPa (3.4 ksi) for 70° skew in exterior girders.

Maximum stress values near midspan govern only for interior girders, and they increase as skew increases—up to 34.5 MPa (5 ksi) at around 30°. Midspan lateral bending stresses rarely govern the response along exterior girders, as observed in Figure 18-B. Since only one governing value along an exterior girder is shown for each bridge, the absence of data points corresponding to midspan regions does not indicate that values above those shown do not exist. Rather, they may be smaller than those at other regions in the same girder for the same analyzed bridge.

For exterior girders, the adopted interpretation of the AASHTO recommendations—8 MPa (1.1 ksi) at span locations and 30 MPa (4.3 ksi) near supports—indicates that the computed envelope stresses remain within acceptable bounds. However, if a uniform limit of 14 MPa (2 ksi) were assumed along the entire bridge length, then stresses at both pier regions and near abutments would significantly exceed the recommended values. Additionally, the present results indicate that the current IDOT approach of neglecting lateral bending stresses below 45° skew may overlook considerable localized lateral bending demands developing near abutments and pier regions.

Bottom-Flange Lateral Bending Stress by Abutment Type

As mentioned in the previous section, overall flange lateral bending stress values for interior and exterior girders remain within those recommended by AASHTO (with an isolated exceedance at 70° for interior girders), except that no stress values are considered when below the skew threshold of 20°. Presence of measurable lateral bending stress below this threshold suggests that certain variables or mechanisms are not explicitly represented in the AASHTO provisions. Consistent with an earlier observation that stresses at small skew occur exclusively near supports, results from Figure 19 indicate that these low-skew lateral bending demands are predominantly associated with IAB configurations. At high skew for interior girders, both stub and integral abutment bridges show approximately the same value in envelopes and IQR. For exterior girders, however, IAB stresses are almost constant in both their envelope and IQR values, while stresses in stub abutment bridges progressively increase with skew. A comparison with Figure 18 indicates that, for exterior girders, lateral bending stress near the abutments controls the response in IABs, while pier-region lateral bending stress governs the response in stub abutment bridges.

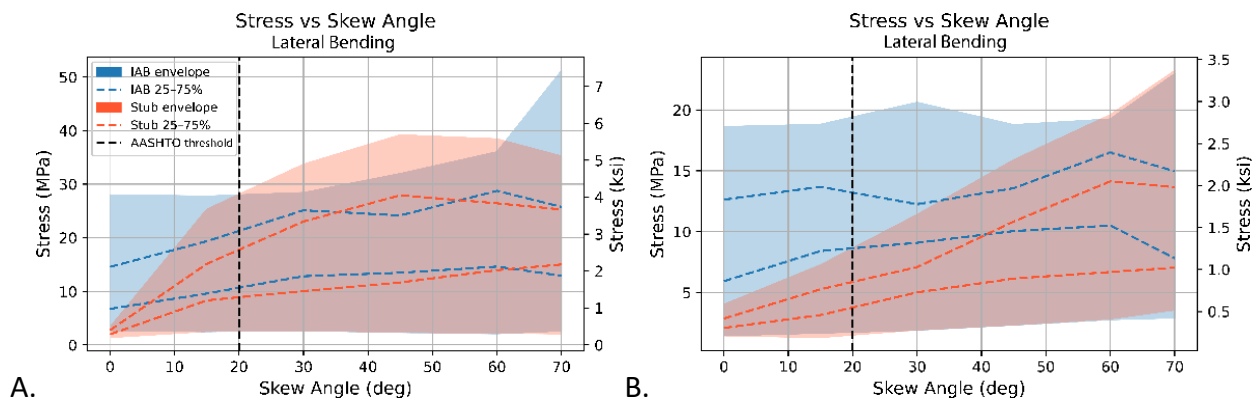


Figure 19. Graph. Envelope and IQR of maximum lateral bending stress vs. skew (by abutment type) for (A) interior girders and (B) exterior girders.

These results indicate that lateral bending stresses in IABs can exceed the levels currently recognized by AASHTO and IDOT at low skew angles, which warrants explicit consideration in design. An examination of bottom flange deformation for zero-skew bridges shows that eccentric load paths relative to adjacent girders, combined with load redistribution through cross-frames, induce transverse deformations in bottom flanges. For IAB configurations, high lateral restraint at the integral abutment—where the bottom flange is effectively restrained against transverse movement and rotation—results in elevated lateral bending stress near the supports. In contrast, stub abutment bridges provide very low bottom flange rotational restraint at the girder ends, significantly reducing lateral bending demands.

Influence of Cross-Frame Configuration on Bottom-Flange Lateral Bending Stress

Preceding sections suggest that flange lateral bending stress is not fully characterized by skew alone, but, rather, could also be influenced by other parameters like girder location and abutment type. Another important factor influencing girder interaction and overall stiffness along a bridge is the cross-frame configuration. To further isolate mechanisms governing lateral bending stress and clarify

sources of variability observed in the previous results, this section examines lateral bending demands as a function of cross-frame type and evaluates the influence of cross-frame spacing.

Figure 20 shows that bridges with parallel (continuous) channel diaphragms are essentially insensitive to skew, with stresses remaining very low across the full skew range. In continuous cross-frame configurations, the transverse load path is not interrupted, so loads do not need to travel through lateral bending. In contrast, both staggered cross-frame configurations exhibit increasing stress with skew angle, suggesting an increase in transverse stiffness for these configurations as bridge skew grows. X-type staggered cross-frames consistently produce larger IQR lateral bending stresses than channel staggered systems. Envelope and IQR stresses for X-staggered configurations are slightly higher than those for channel-staggered configurations on exterior girders, independent of skew angle. For interior girders, IQR values are consistently 60%–80% higher for X-staggered configurations, also independent of skew. The relative increase associated with X-staggered systems compared to channel-staggered systems is evident for both girder locations, but absolute stress differences are larger for interior girders. This more pronounced increase of stress in interior girders for bridges with X cross-frames is most likely attributable to effects of the transverse load path, which engages interior girders more directly as loads travel between obtuse corners. As skew angle increases, transverse force transfer between girders becomes more significant and the higher stiffness of X-type cross-frames attracts a larger share of this load. The resulting increase in force amplifies lateral bending demands on the interior girders.

Figure 20 helps explain the variability observed in Figure 24: Upper values mostly correspond to X-staggered configurations, while lower values mostly correspond to channel-staggered configurations. Consistently lower values associated with channel-staggered configurations suggest that current lateral bending requirements may be conservative for these systems and could warrant further evaluation.

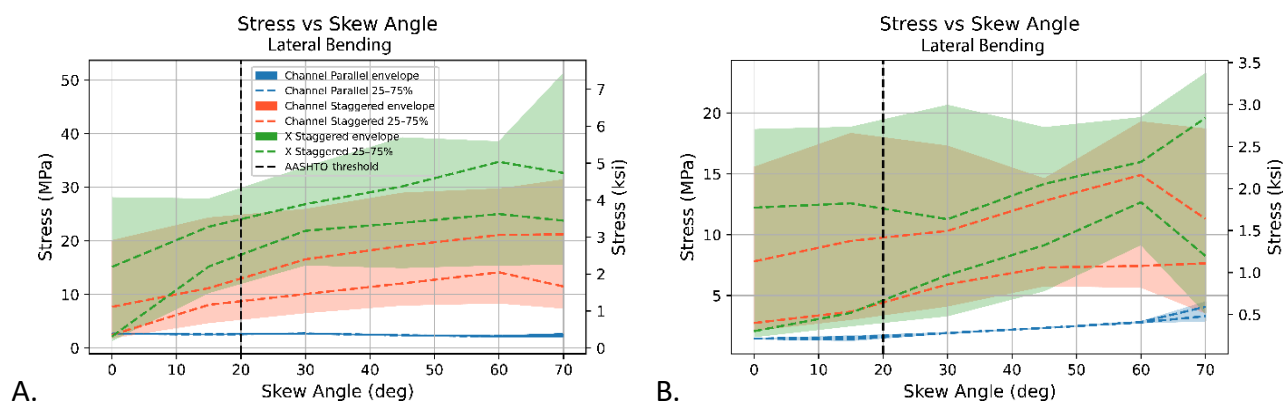


Figure 20. Graph. Envelope and IQR of maximum lateral bending stress vs. skew (by cross-frame type) for (A) interior girders and (B) exterior girders.

Effects of AASHTO Minimum First Cross-Frame Spacing Compliance

This section examines the adequacy of AASHTO provisions intended to limit transverse load transfer near supports. Figure 21 shows the relationship between stress demand and bottom flange unrestrained length from the support (denoted as cross-frame spacing), evaluated against the AASHTO minimum first-cross-frame spacing requirement of $4b_f$. Each data point represents a governing lateral bending stress location, and the corresponding bottom flange unrestrained length is measured at the same longitudinal position along the girder. The applicable limit is determined using the bottom flange width at that location, ensuring the comparison reflects local section geometry rather than a nominal or average value. The reported cross-frame spacing corresponds to the distance along the girder bottom flange between the abutment and the first cross-frame (at either side of the girder) at the cross-section where the maximum lateral bending stress is recovered.

Only data points corresponding to bridges where maximum lateral bending stress locations occur within regions where the AASHTO $4b_f$ limit is applicable are included—namely, between the supports and the first cross-frame. Data points from maximum stress values at interior locations (spans and piers), where the specification does not impose a spacing restriction, are excluded from this plot, which enables direct assessment of the prevalence and severity of exceedances while maintaining consistency with the intended scope of the specification.

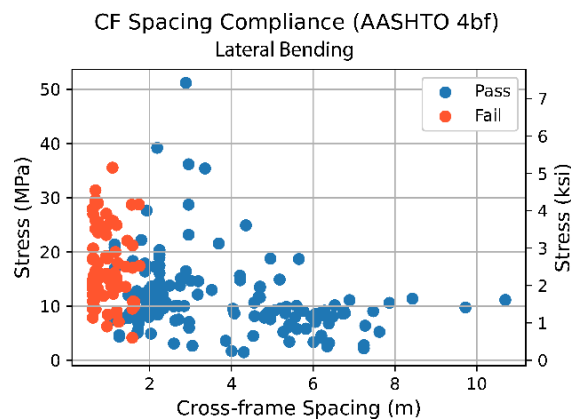


Figure 21. Graph. AASHTO first cross-frame spacing compliance: stress vs. bottom flange unrestrained length.

Configurations satisfying the $4b_f$ requirement are generally associated with lower lateral bending stress demands (Figure 21). In contrast, higher stress magnitudes—approaching 34.5 MPa (5 ksi)—are more frequently seen among cases that do not satisfy the $4b_f$ criterion.

Figure 22 further examines the influence of first cross-frame spacing compliance with respect to skew angle, where maximum values have again been filtered to correspond only to those near abutments (where the $4b_f$ limit is applicable). Independent of skew angle, IQR values that fail to comply with the minimum spacing requirement are typically larger.

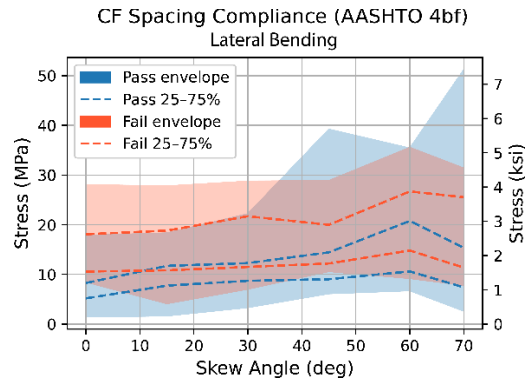


Figure 22. Graph. AASHTO first cross-frame spacing compliance: stress vs. skew angle.

These results suggest that compliance with the $4b_f$ spacing provision contributes to a reduction in girder flange live load–induced lateral bending stresses near the abutments, consistent with the intent of AASHTO to limit development of stiff transverse load paths that attract large force transfer into skewed supports. In the context of current IDOT provisions, these observations are particularly relevant for IABs, since Section 3.3.22 of the IDOT *Bridge Manual* recommends placement of the first cross-frame 0.6 m (2 ft) into the span from the inside face of the concrete diaphragm to ensure girder stability during construction. The present results suggest that placement of cross-frames very near supports can influence development of transverse load paths and associated lateral bending demands under service loading conditions.

Bottom-Flange Lateral Bending Stress at Critical Sections

While the preceding sections evaluated locations where maximum lateral bending stresses occur, these locations do not necessarily coincide with the cross-sections governing maximum total girder stress. From a design perspective, it is therefore important to assess the magnitude of lateral bending stresses specifically at sections controlling total stress demand, since these locations ultimately govern girder design and evaluation. This comparison also provides insight into how much of the previously discussed lateral bending behavior directly contributes to critical girder demand, particularly near supports where large localized lateral bending stresses were observed.

Evaluation of lateral bending stresses at cross-sections governing maximum total stress indicates that the qualitative trends identified previously remain valid, although stress magnitudes are reduced relative to the maximum lateral bending values occurring anywhere along the bridge. For interior girders, lateral bending stresses at governing total stress sections range from approximately 26 MPa (3.8 ksi) at low skew to 34.5 MPa (5 ksi) at high skew, representing an approximate net reduction of 7 MPa (1 ksi) relative to the previously reported envelope stresses. Exterior girders show larger relative reductions, with governing-section stresses generally below 18 MPa (2.5 ksi), corresponding to net reductions of roughly 3.5–7 MPa (0.5–1 ksi) compared to maximum lateral bending stresses anywhere along the bridge. The reduction occurs primarily near support regions, indicating that sections experiencing the largest lateral bending demands do not always coincide with sections governing maximum total stress. Nevertheless, the results also show that, particularly for integral abutment bridges, considerable lateral bending stresses remain present even at governing total stress locations, including low-skew configurations below the current AASHTO applicability threshold.

CHAPTER 4: BRIDGE RESPONSE UNDER THERMAL LOADS— FIELD MONITORING

The overall field-monitoring campaign, monitored bridges, and instrumentation layouts are discussed in Volume I (Zhou et al. 2026b) and revisited in Chapter 1 of this document. Field behavior of the monitored bridges during the deck placement stage, as well as under truck tests, are discussed in Volume II (Zhou et al. 2026a). This chapter focuses on the behavior of the monitored bridges under long-term loads, using data from September 2021 through December 2025 collected at 0.5 Hz. Over this period, the minimum mean temperature recorded was -23°C , while the maximum mean temperature recorded was 37°C , for a total temperature range of 60°C , recorded through the built-in thermistors of each gauge. The temperature profile and extreme values are presented in Figure 6.

As in previous parts of this study, stress data are inferred from strain gauge readings, assuming elastic behavior of steel I-girders and cross-frames, and decomposed using the stress decomposition scheme presented in Volume I (Zhou et al. 2026b), which is also briefly explained in previous chapters of this document. Data are presented both in raw (individual stress readings from each gauge) and decomposed formats (different girder actions: strong-axis bending, weak-axis bending, and axial stress). Since the accuracy of the decomposition is dependent on the estimation of the elastic neutral axis (ENA) location, field-based neutral axes were calculated for the monitored cross-sections using 20 Hz data from random truck passes at different points in time. Neutral axis locations were estimated using readings from top and bottom flange gauges at the instant of maximum stress during each truck event (zero crossing of the stress profile). The stresses were initialized at zero for each truck event to minimize the influence of long-term axial effects on the ENA. Comparisons between field- and hand-calculated (based on the assumed geometry) ENAs indicated generally good agreement for the stub abutment bridge, while larger discrepancies were observed for the integral abutment bridge, particularly for exterior girders, suggesting a more complex stress distribution and possible structural engagement of parapets. Consequently, the field-calculated ENAs were adopted in the stress decomposition scheme presented herein.

Clear trends were observed in the recorded data as a function of temperature variation. Short-term behavior differed significantly from long-term behavior due to temperature changes, possibly showing the effects of uneven thermal variations (thermal gradients) on the bridge. These thermal variations can be observed through an analysis of the temperature data recorded from the strain gauges built-in thermistors. Temperature data recorded at Mattis-57 are used as an example.

Figure 23 shows temperature data collected from bottom and top flanges, from interior and exterior girders as well as from gauges embedded in the concrete deck at Mattis-57. Gradients exist primarily during warmer months (Figure 23-B), whereas temperature data collected during colder months (Figure 23-C) are relatively similar for all recording gauges. During warmer periods, gradients exist both vertically through the depth of the superstructure and horizontally across its width. For interior girders (continuous lines), higher temperatures are consistently recorded from top flange gauges. On exterior girders, on the other hand, bottom flange gauges experience higher temperatures near

noon, with a distinctive peak attributed to uneven heating under direct sunlight, which has been discussed by Zhou et al. (2024).

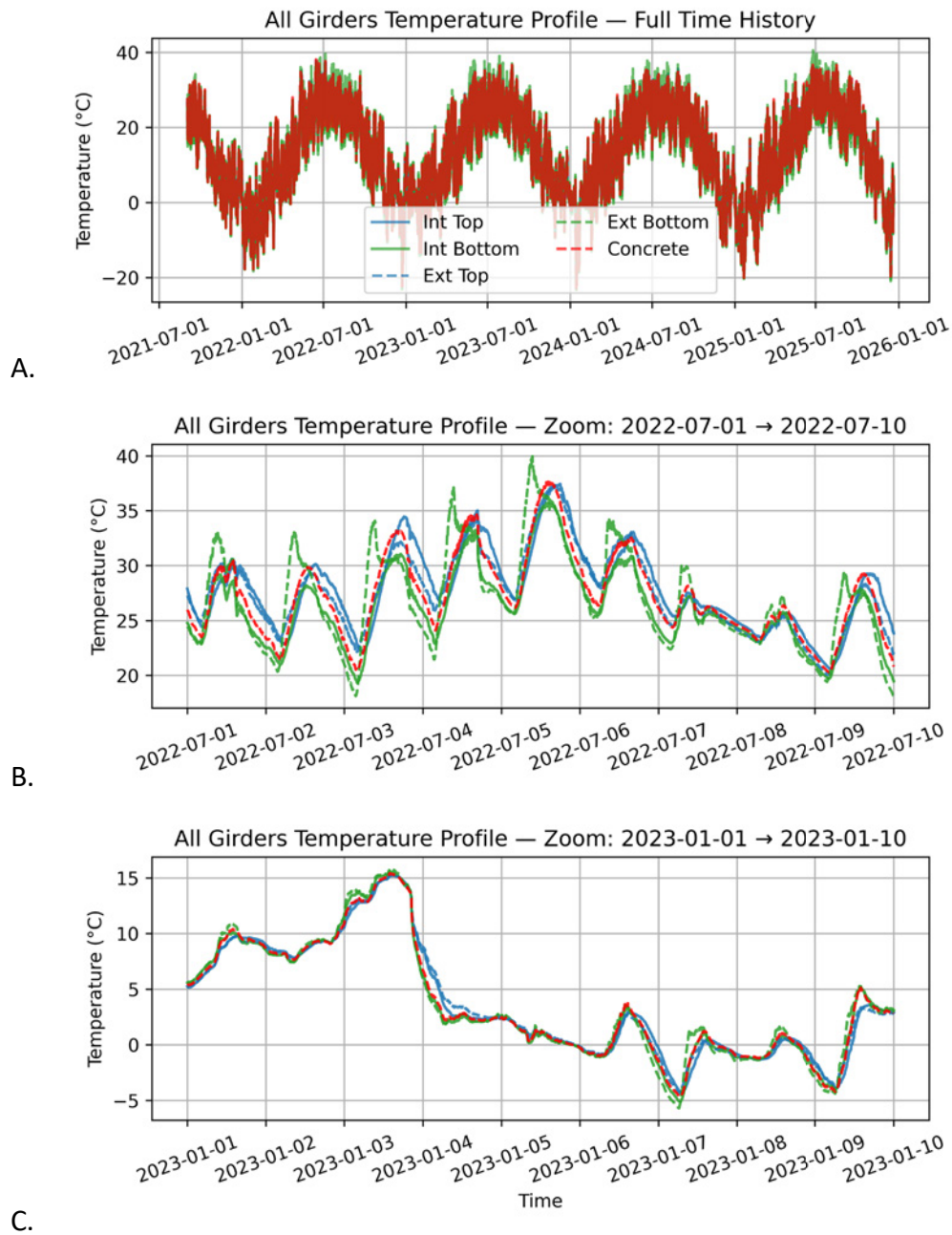


Figure 23. Graph. Temperature data collected from Mattis-57: (A) full monitoring period, (B) warmer months, and (C) colder months.

The effects of the observed gradients are discussed in subsequent sections of this chapter. Additionally, selected field data are presented with emphasis on comparing the behavior of stub abutment and integral abutment bridges. The presented information is organized into the following categories: raw stress data, strong-axis bending stresses, weak-axis bending stresses, and bridge

movements. Bridge movement data focus on measurements obtained from crackmeters and tiltmeters.

BACKGROUND

This chapter examines the long-term thermal behavior of the two monitored bridges: a stub abutment bridge and an integral abutment bridge. The two structures share similar geometric characteristics, including span configuration, overall bridge length, and skew angle, and are in close proximity. As a result, they are subjected to essentially the same environmental conditions, including comparable ambient temperature variations. This provides a unique opportunity to directly assess differences in structural response attributable primarily to the abutment configuration.

The evaluation of long-term thermal effects is particularly important for bridges with integral abutments. Unlike conventional stub abutment bridges, IABs do not incorporate expansion joints and instead accommodate thermal movements through interaction with the supporting soil and foundation system. This soil-structure interaction introduces additional sources of stiffness and restraint, leading to a more complex structural response under thermal loading.

For skewed bridges, this behavior is further complicated by geometric effects. Thermal expansion and contraction induce longitudinal movements that, when combined with skew, generate in-plane rotations of the bridge superstructure. These rotations arise from the eccentricity between the backfill reactions at the abutments, which act normal to the abutment face (Okumus et al. 2018). As a result, skewed IABs may experience coupled translational and rotational responses, influencing load distribution among girders and increasing the potential for lateral bending effects.

By leveraging the similarities between the monitored bridges, the field data presented in this chapter enables a direct comparison of the influence of abutment type and skew on long-term thermal behavior of steel I-girder bridges.

Previous long-term field-monitoring studies conducted on bridges in Illinois (LaFave et al. 2017, 2021) reported significant deviations in strain gauge readings, particularly in bottom-flange gauges, with deviations often initiating during colder months. These deviations were interpreted as potential indicators of structural behavior and were hypothesized to result from mechanisms such as out-of-plane bending, leading to localized yielding or other time-dependent effects associated with abutment restraint, foundation interaction, or deck cracking. Interpretation of these observations, however, was inherently limited by the instrumentation scheme, which included only a single gauge per flange location. This configuration prevented decomposition of stresses into individual girder actions (e.g., strong-axis bending, weak-axis bending, and axial effects) and restricted evaluation to direct interpretation of raw strain measurements at isolated locations.

The present study revisited these previously reported behaviors using a more comprehensive instrumentation approach, which allowed for decomposition of stresses, as discussed in previous sections of this document. Within this framework, deviations similar to those reported by LaFave et al. (2017, 2021) were also observed in the current dataset, frequently initiating during colder periods and often exhibiting progressive increases in tensile strain. Comparisons among gauges within the

same cross-section revealed that such deviations were not accompanied by consistent changes in the overall stress distribution. Specifically, when a deviation was observed in one gauge (e.g., on one side or surface of the flange), the remaining gauges at that cross-section—particularly those on the opposite side or at corresponding flange locations—did not exhibit similar changes in magnitude or trend. This lack of consistency is incompatible with expected structural behavior, where stress changes at a given cross-section should produce a coherent and mechanically consistent response across all gauges at that location.

To further investigate the origin of these deviations, additional strain gauges were installed adjacent to selected existing gauges that exhibited anomalous behavior. The new gauges were placed close to the original gauges, allowing for direct comparison of measurements under identical structural and environmental demands. The newly installed gauges exhibited consistent trends that aligned with the expected structural response and with the behavior of other gauges at the same cross-section. In contrast, the original gauges continued to show deviations from typical trends.

This direct side-by-side comparison confirmed that the observed deviations were not representative of structural behavior but indicative of strain gauge malfunction. Accordingly, readings exhibiting such behavior were classified as spurious and excluded from further analysis and interpretation. Additional details regarding the identification, verification, and handling of these readings are provided in Appendix A.

I-GIRDER RAW STRESS

This section presents the raw stress data directly obtained from individual strain gauges, without decomposition into specific girder actions. These results provide an overall view of the measured response and serve as a reference for identifying general trends, seasonal variations, and differences between the monitored bridges. Selected plots are used to illustrate maximum responses. The full set of raw stress data is presented in Appendix A.

Mattis-74 (Stub)

Table 3 summarizes key stress measures from Mattis-74. Maximum, minimum, and overall range of stress values over the monitoring period are presented for gauges from which these extreme values were recovered. The results are organized per location along the bridge length (pier or span for Mattis-74) and per girder type (exterior or interior). A maximum stress range of 40 MPa (5.8 ksi) is recorded at pier locations (interior girder), whereas a maximum stress range of 47.5 MPa (6.9 ksi) is recorded near span locations (exterior girder). The reported stress ranges should be interpreted as indicative of the overall variation in stress demand over the monitoring period, with approximately one-half of the range corresponding to the magnitude of the maximum tensile or compressive stress relative to the mean response. This overall variation encompasses stresses developed due to both uniform temperature changes and thermal gradients.

Table 3. Summary of Maximum Stress Data from Girder Cross-Section Gauges—Mattis-74

Cross-Section	Location	Girder Type	Gauge	Min. Stress MPa (ksi)	Temp. at Min. (°C)	Max. Stress MPa (ksi)	Temp. at Max. (°C)	Stress Range MPa (ksi)
G1 Pier	pier	exterior	BEb	-10 (-1.5)	33.8	28 (4.1)	-15.8	38 (5.5)
G3 Pier	pier	interior	BEb	-8.5 (-1.2)	30.6	31.5 (4.5)	-21.5	40 (5.8)
G1 N. Span	span	exterior	BWb	-9 (-1.3)	25.2	38.5 (5.6)	24.7	47.5 (6.9)
G5 S. Span	span	interior	TW	-9.5 (-1.4)	32.2	31.5 (4.5)	-18.1	40.5 (5.9)

The response history data of all gauges at each cross-section in Table 3 are presented in Figures 24 through 27.

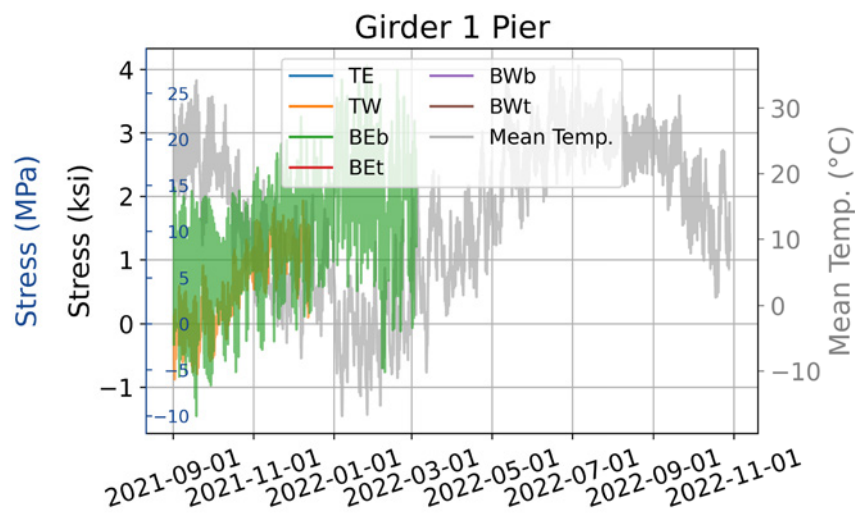


Figure 24. Graph. Stress data collected from Girder 1 Pier.

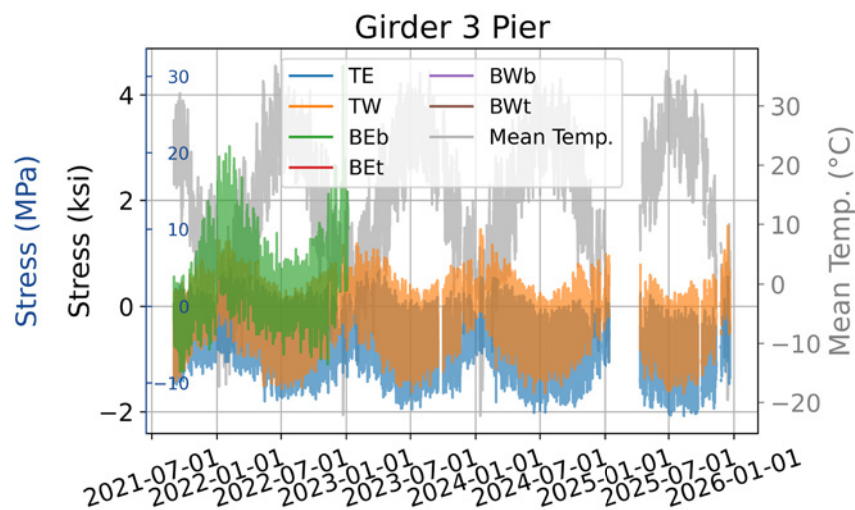


Figure 25. Graph. Stress data collected from Girder 3 Pier.

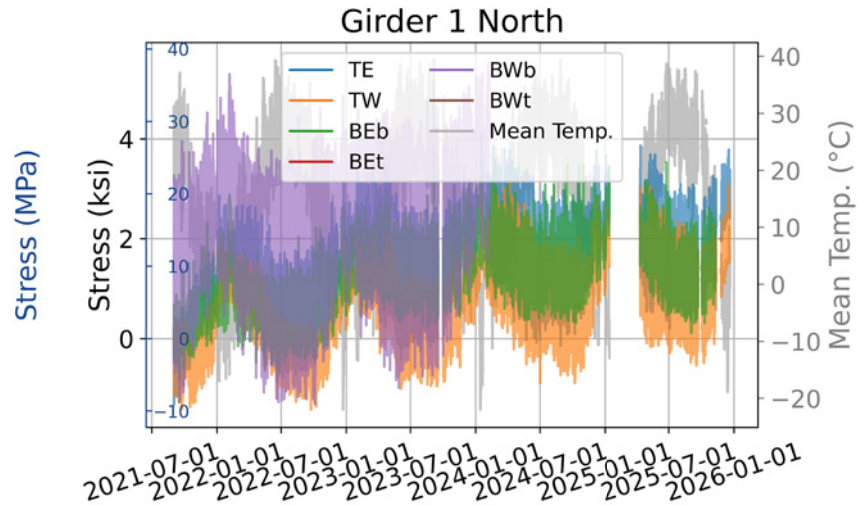


Figure 26. Graph. Stress data collected from Girder 1 North Span.

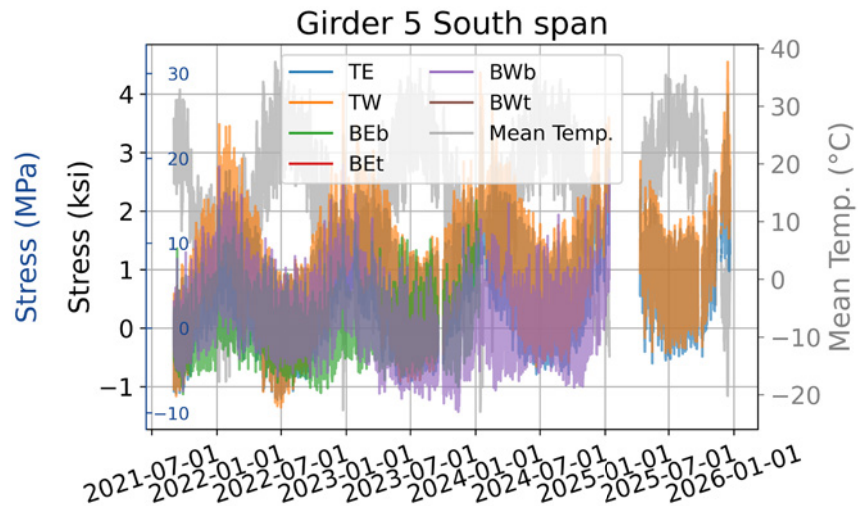


Figure 27. Graph. Stress data collected from Girder 5 South Span.

Figure 28 shows temperature versus stress plots for the individual gauges presented in Table 3. Most gauges' readings demonstrate a clear correlation of stresses with change in temperature.

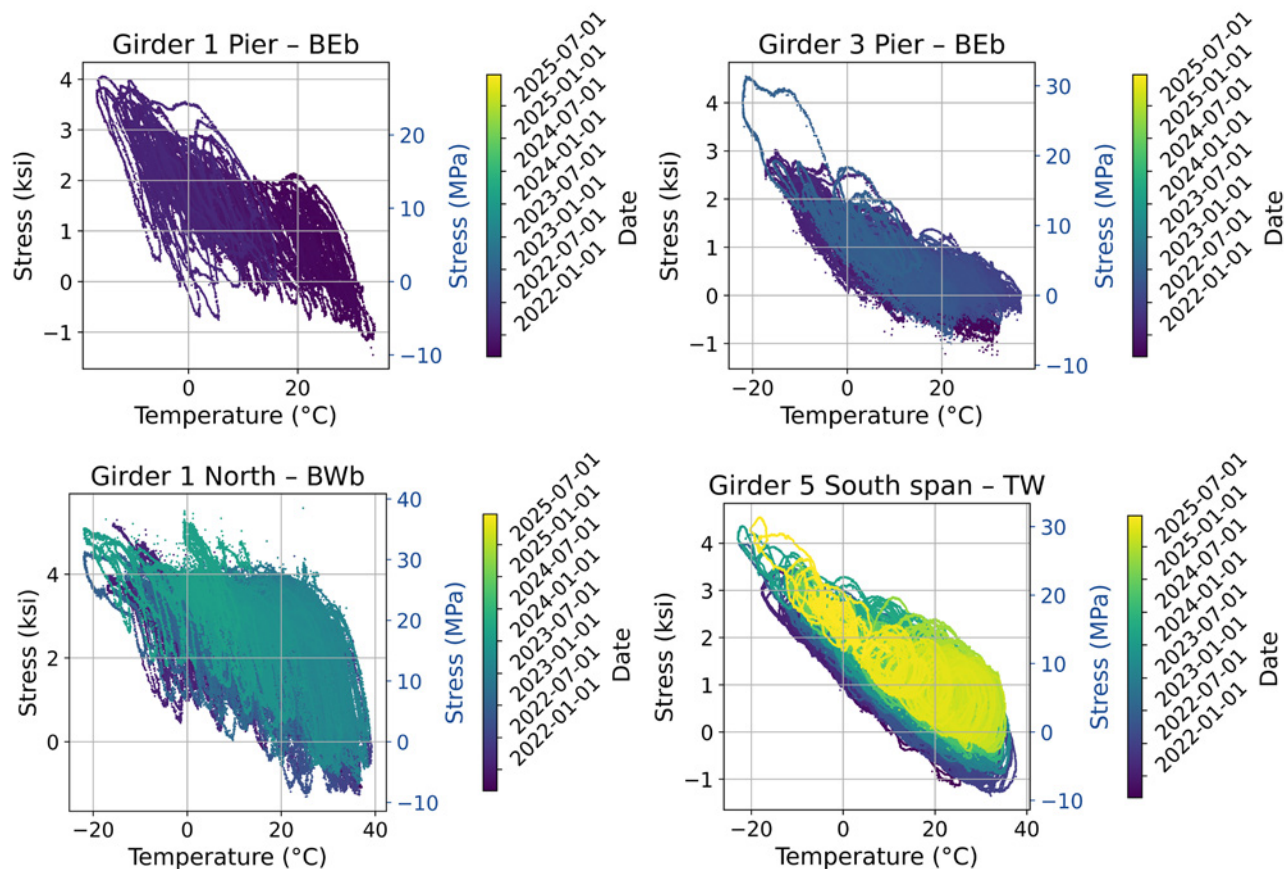


Figure 28. Graph. Temperature vs. stress data collected from critical gauges.

The temperature versus stress plots reveal a noticeable scatter around the dominant long-term thermal trend (overall slope). As discussed earlier in this chapter, this behavior is most likely associated with non-uniform thermal gradients that produce short-term structural response. These thermal gradients are expected to develop primarily during daily temperature fluctuations, where temperature changes occur more rapidly than the gradual variations associated with seasonal thermal cycles. While the primary focus of the present section is the overall long-term thermal behavior of the bridges, the magnitude and characteristics of this short-term scattered response are evaluated in detail in subsequent sections through a regression-based statistical analysis.

Mattis-57 (IAB)

Table 4 summarizes the key measures from Mattis-57. Maximum, minimum, and overall range of stress values over the monitoring period are presented for gauges from which these extreme values were recovered. The results are separated per location along the bridge length (pier, span, or abutment for Mattis-57) and per girder type (exterior or interior). Maximum stress ranges for the abutment and span regions seem to be larger than for the stub abutment bridge, particularly on exterior girders, reaching up to 107 MPa (15.5 ksi) and 52 MPa (7.6 ksi), respectively. This overall variation encompasses stresses developed due to both uniform temperature changes and thermal gradients.

Note that the recorded field data correspond to discrete instrumented locations throughout each bridge and do not necessarily represent the locations of maximum stresses within the superstructure. In addition, differences in instrumentation layouts, monitored cross-sections, and girder locations between the two bridges make direct comparisons of the measured extreme stress ranges inherently difficult. Consequently, the comparisons presented herein should be interpreted primarily as observations of the monitored response at the instrumented locations rather than definitive indicators of the global maximum bridge response. This limitation is addressed through the parametric finite element analyses presented in Chapter 5, in which stresses are evaluated continuously throughout the bridge girders, allowing the identification of critical locations and maximum responses anywhere within the superstructure, using models that have been validated under thermal demands, as presented in Chapter 2.

Table 4. Summary of Maximum Stress States from Girder Cross-Section Gauges—Mattis-57

Cross-Section	Location	Girder Type	Gauge	Min. Stress MPa (ksi)	Temp. at Min. (°C)	Max. Stress MPa (ksi)	Temp. at Max. (°C)	Stress Range MPa (ksi)
G8 N. Abut.	abut.	exterior	BEb	−54.5 (−7.9)	38.5	52.5 (7.6)	−21.6	107 (15.5)
G5 S. Abut.	abut.	interior	BEb	−30 (−4.3)	34.9	43 (6.2)	−22.9	73 (10.6)
G8 Pier S.	pier	exterior	BWb	−13 (−1.9)	30.9	15.5 (2.3)	3.1	29 (4.2)
G5 Pier N.	pier	interior	TE	−13.5 (−2)	31.4	24 (3.5)	−21.9	37.5 (5.5)
G8 S. Span	span	exterior	BEb	−48 (−7)	31.1	30.5 (4.4)	−16.6	78.5 (11.4)
G6 S. Span	span	interior	BEt	−11 (−1.6)	35.0	41.5 (6)	−22.3	52.5 (7.6)

The response history data of all gauges at each cross-section in Table 4 are presented in Figure 29 through Figure 34.

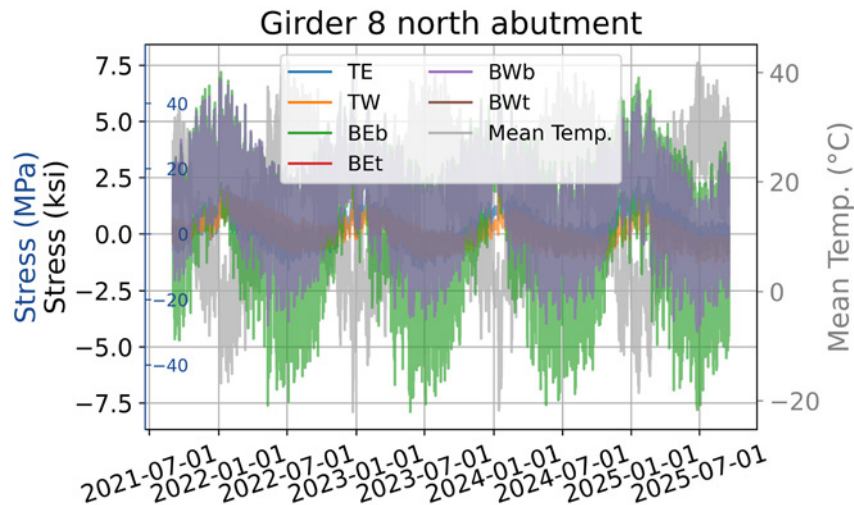


Figure 29. Graph. Stress data collected from Girder 8 North Abutment.

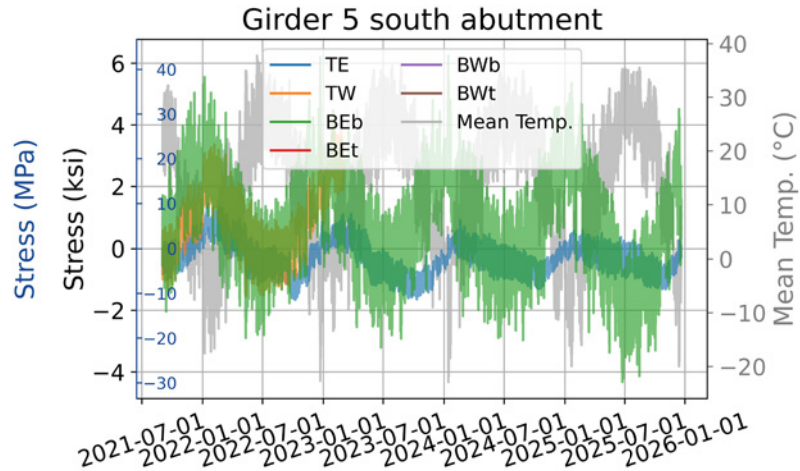


Figure 30. Graph. Stress data collected from Girder 5 South Abutment.

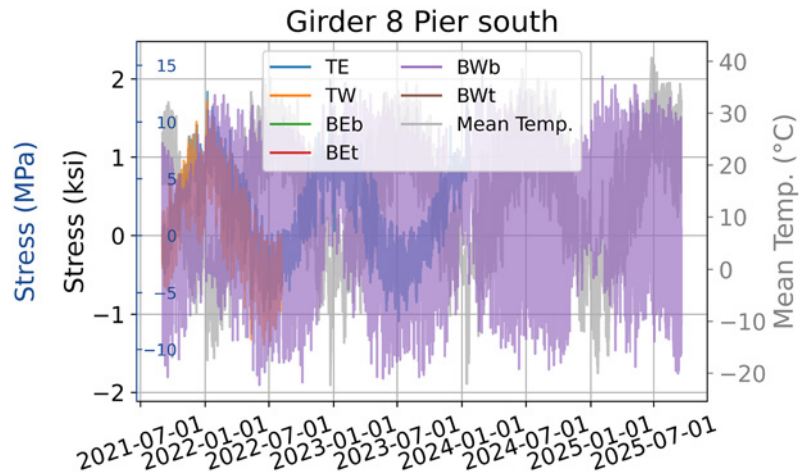


Figure 31. Graph. Stress data collected from Girder 8 Pier.

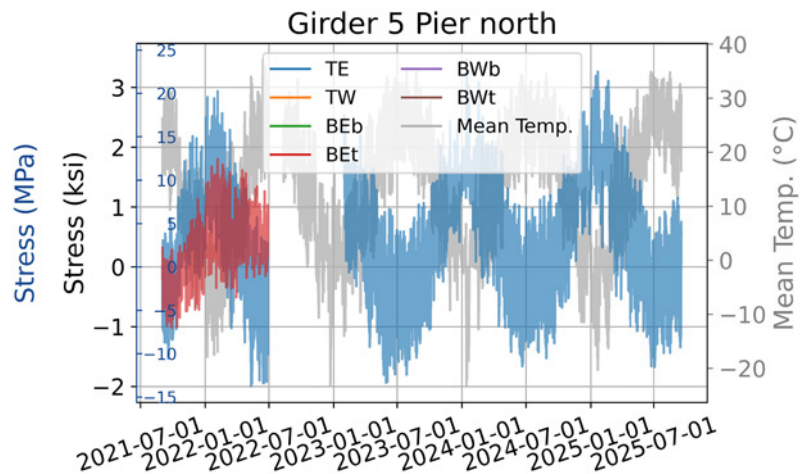


Figure 32. Graph. Stress data collected from Girder 5 Pier North.

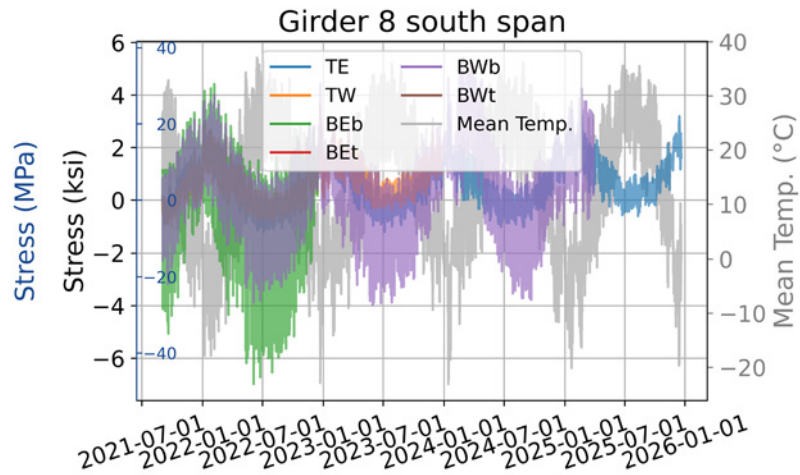


Figure 33. Graph. Stress data collected from Girder 8 South Span.

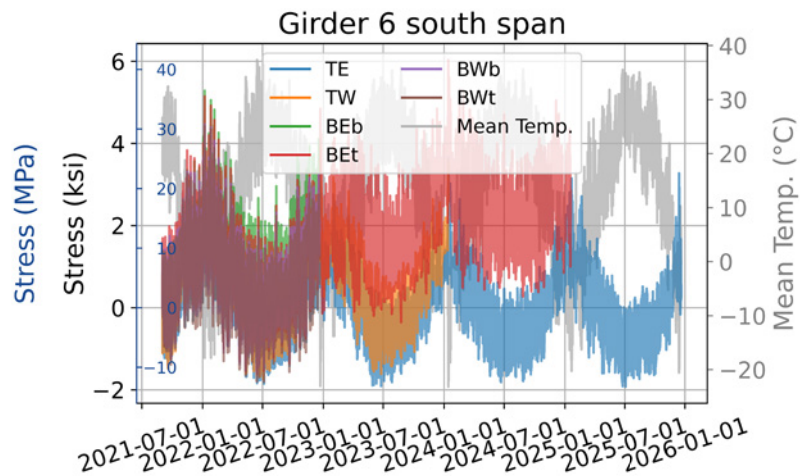


Figure 34. Graph. Stress data collected from Girder 6 South Span.

Figure 35 shows temperature versus stress plots for the individual gauges presented in Table 4.

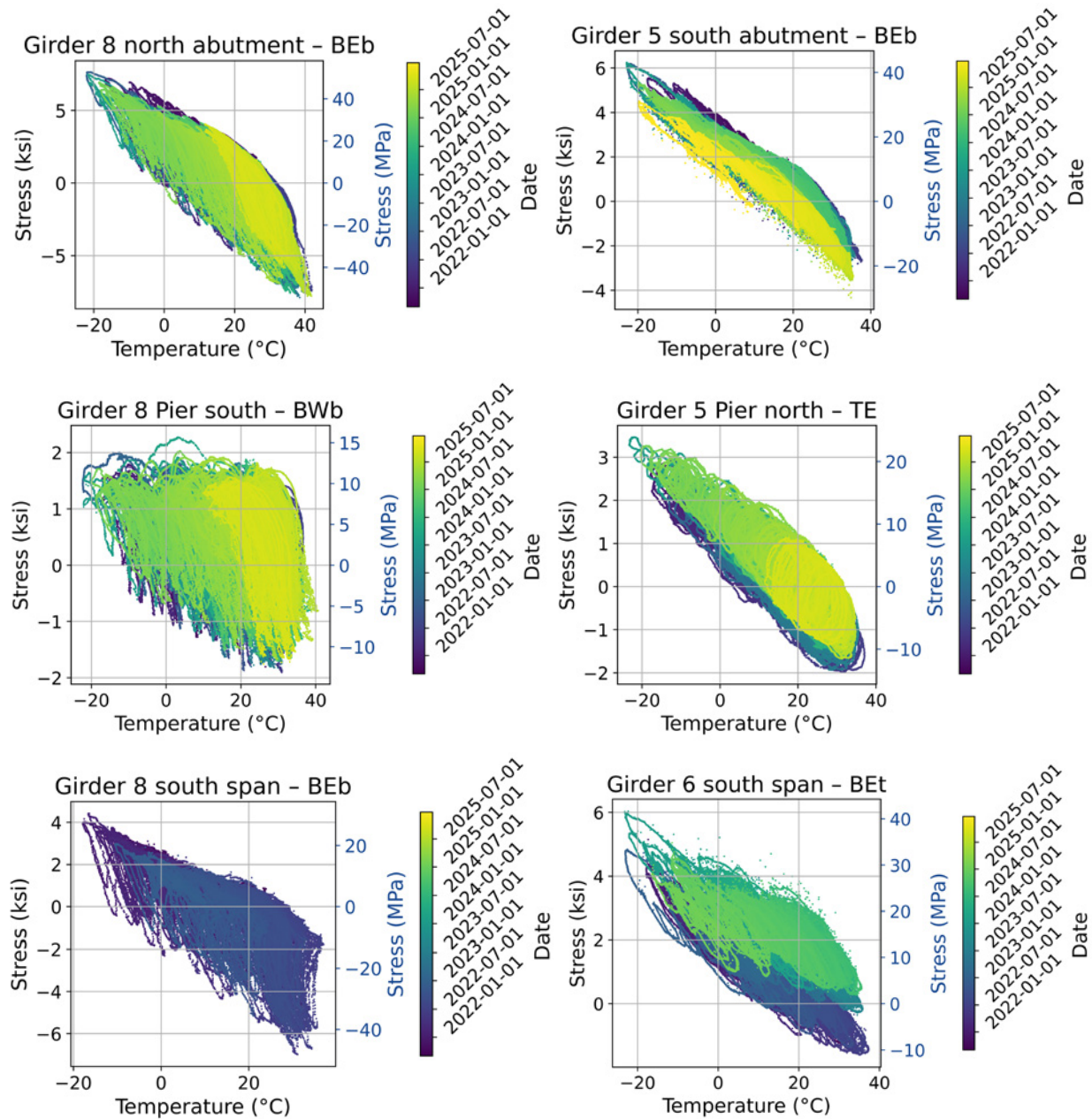


Figure 35. Graph. Temperature vs. stress data collected from critical gauges.

I-GIRDER STRONG-AXIS BENDING STRESSES

This section focuses on the strong-axis bending component of girder response. The presented results highlight how thermal effects influence the dominant flexural response and how this behavior varies between stub abutment and integral abutment configurations. Selected plots are used to illustrate maximum responses, and the full set of strong-axis bending stress data is presented in Appendix A.

Mattis-74 (Stub)

Table 5 summarizes maximum and minimum strong-axis bending measures collected from Mattis-74. The results are separated per location along the bridge length (pier or span for Mattis-74) and per girder type (exterior or interior). A stress range up to 42.5 MPa (6.2 ksi) is observed for cross-sections near the span on exterior girders, while pier regions remain with lower values, reaching 21MPa (3 ksi) on interior girders. The overall stress ranges presented herein encompass stresses developed due to both uniform temperature changes and thermal gradients.

Table 5. Summary of Strong-Axis Bending Stress Data from Girder Cross-Section Gauges—Mattis-74

Cross-Section	Location	Girder Type	Component	Min. Stress MPa (ksi)	Temp. at Min. (°C)	Max. Stress MPa (ksi)	Temp. at Max. (°C)	Stress Range MPa (ksi)
G5 Pier	pier	interior	SA bottom	-10.5 (-1.5)	20.5	10.5 (1.5)	2.0	20.5 (3)
G4 Pier	pier	interior	SA top	-4 (-0.6)	-1.6	3 (0.4)	21.0	7 (1)
G1 N. Span	span	exterior	SA bottom	-14 (-2.1)	26.5	28.5 (4.1)	24.7	42.5 (6.2)
G1 N. Span	span	exterior	SA top	-7 (-1)	24.7	3.5 (0.5)	26.5	10.5 (1.5)
G5 S. Span	span	interior	SA bottom	-18 (-2.6)	-0.9	12 (1.8)	30.5	29.5 (4.3)
G4 N. Span	span	interior	SA top	-4 (-0.5)	23.5	3 (0.5)	-20.4	7 (1)

The response history and temperature versus stress data (bottom flange) at each cross-section in Table 5 is presented in Figure 36 and Figure 37, respectively.

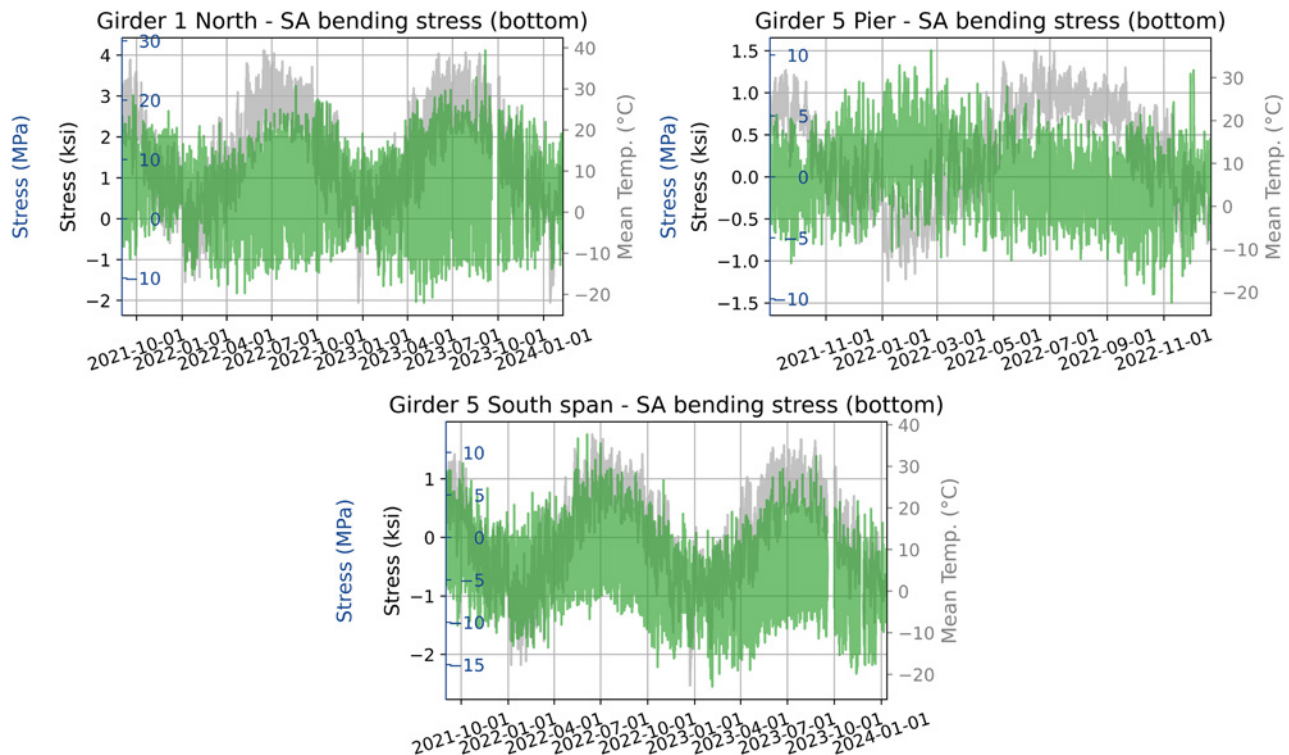


Figure 36. Graph. Strong-axis bending stress data—bottom flange.

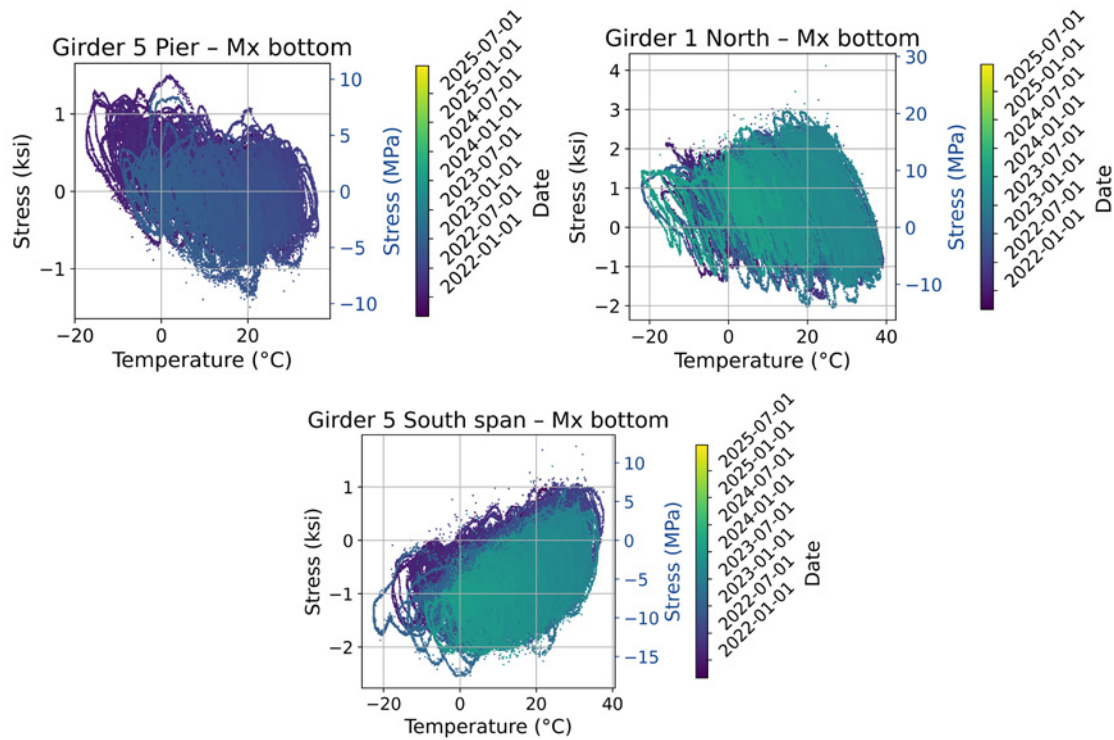


Figure 37. Graph. Temperature vs. strong-axis bending stress data—bottom flange.

Mattis-57 (IAB)

Table 6 summarizes maximum and minimum strong-axis bending measures from Mattis-57. The results are separated per location along the bridge length (pier, span, or abutment for Mattis-57) and per girder type (exterior or interior). Slightly lower stress ranges than for the stub abutment bridge are observed both at the span and pier locations. The overall stress ranges presented herein encompass stresses developed due to both uniform temperature changes and thermal gradients.

Table 6. Summary of Strong-Axis Bending Stress Data from Girder Cross-Section Gauges—Mattis-57

Cross-Section	Location	Girder Type	Component	Min. Stress MPa (ksi)	Temp. at Min. (°C)	Max. Stress MPa (ksi)	Temp. at Max. (°C)	Stress Range MPa (ksi)
G8 N. Abut.	abut.	exterior	SA bottom	-29 (-4.2)	37.4	28 (4.1)	-14.1	57 (8.3)
G8 N. Abut.	abut.	exterior	SA top	-10.5 (-1.6)	-14.1	11 (1.6)	37.4	22 (3.2)
G7 S. Abut.	abut.	interior	SA bottom	-23.5 (-3.4)	31.2	18.5 (2.7)	-8.0	42 (6.1)
G6 S. Abut.	abut.	interior	SA top	-1.5 (-0.3)	-2.8	2 (0.3)	31.4	4 (0.6)
G6 Pier S.	pier	interior	SA bottom	-9.5 (-1.4)	19.7	9 (1.3)	32.5	18.5 (2.7)
G6 Pier S.	pier	interior	SA top	-1.5 (-0.2)	32.5	1.5 (0.2)	19.7	2.5 (0.4)
G8 S. Span	span	exterior	SA bottom	-29 (-4.2)	31.0	12 (1.8)	3.9	41 (6)
G8 S. Span	span	exterior	SA top	-3 (-0.5)	3.9	7.5 (1.1)	31.0	11 (1.6)
G6 S. Span	span	interior	SA bottom	-9.5 (-1.4)	19.1	18 (2.5)	16.4	27 (3.9)
G6 S. Span	span	interior	SA top	-4.5 (-0.6)	16.4	2.5 (0.3)	19.1	7 (1)

The response history and temperature versus stress data (bottom flange) at each cross-section in Table 6 is presented in Figure 38 and Figure 39, respectively.

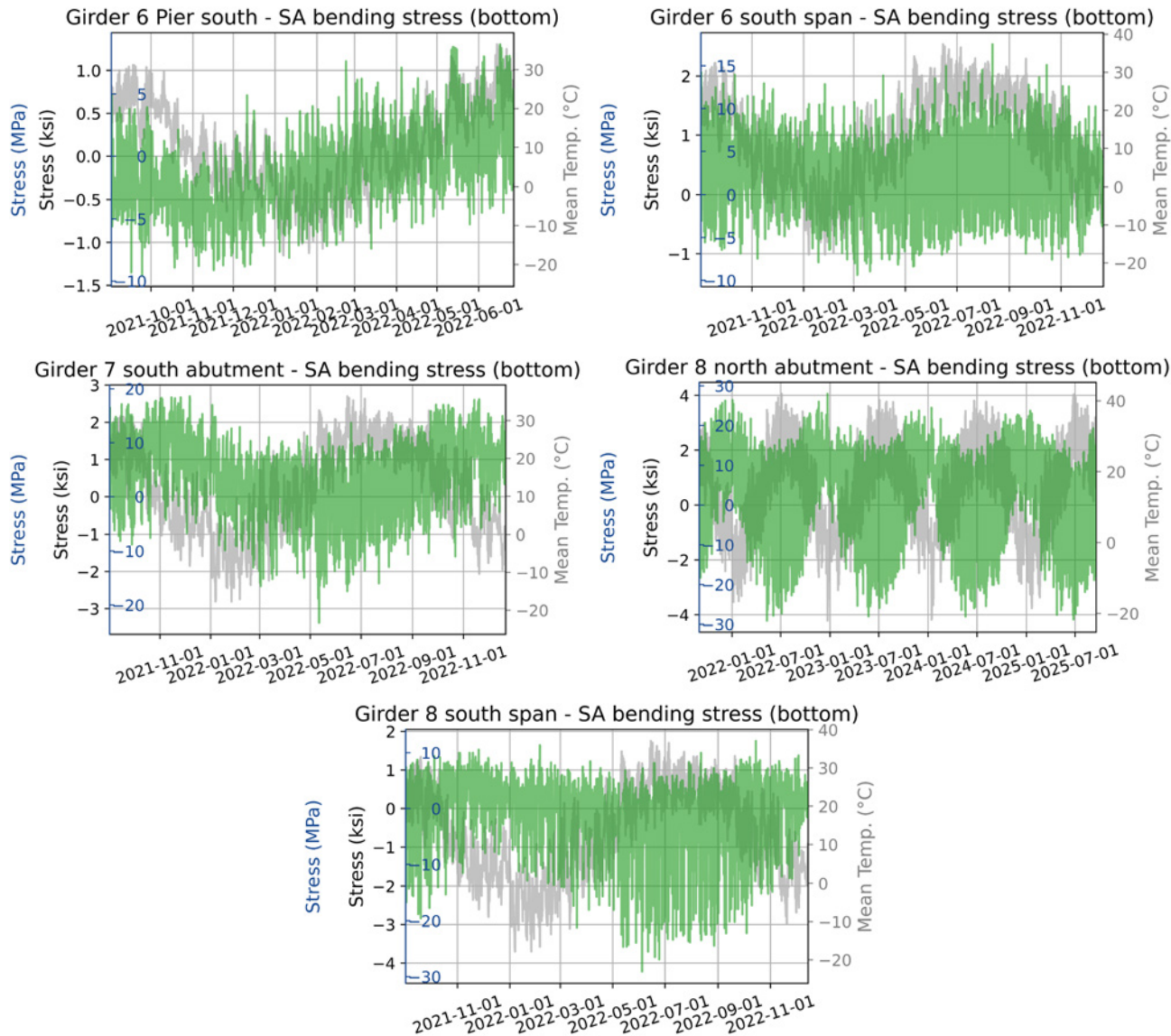


Figure 38. Graph. Strong-axis bending stress data—bottom flange.

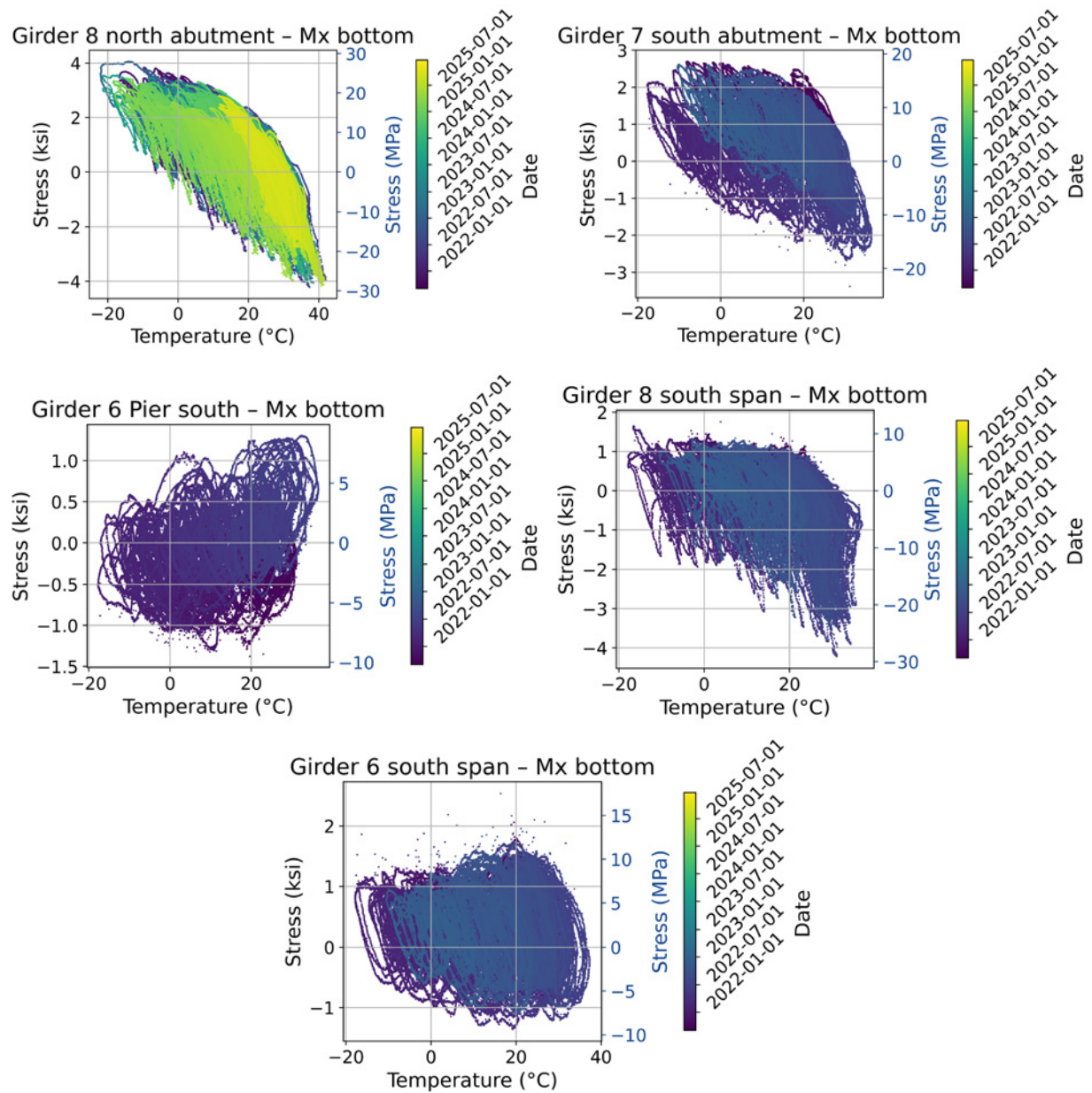


Figure 39. Graph. Temperature vs. strong-axis bending stress data—bottom flange.

I-GIRDER BOTTOM-FLANGE LATERAL BENDING STRESS

This section examines the lateral bending stress component, which is primarily associated with transverse load redistribution and geometric effects such as skew. Selected plots are used to illustrate maximum responses. The full set of bottom-flange lateral bending stress data is presented in Appendix A.

Mattis-74 (Stub)

Table 7 summarizes maximum and minimum bottom flange weak-axis (lateral) bending measures from Mattis-74. The results are separated per location along the bridge length (pier or span for Mattis-74) and per girder type (exterior or interior). The maximum lateral bending stress range recorded for Mattis-74 is 17 MPa (2.4 ksi) and occurs near the span on an exterior girder. The overall stress ranges presented herein encompass stresses developed due to both uniform temperature changes and thermal gradients.

Table 7. Summary of Weak-Axis Bending Stress Data from Girder Cross-Section Gauges—Mattis-74

Cross-Section	Location	Girder Type	Component	Min. Stress MPa (ksi)	Temp. at Min. (°C)	Max. Stress MPa (ksi)	Temp. at Max. (°C)	Stress Range MPa (ksi)
G4 Pier	pier	interior	WA bottom	-3 (-0.4)	20.3	3 (0.4)	15.1	5.5 (0.8)
G1 N. Span	span	exterior	WA bottom	-11.5 (-1.7)	25.6	5.5 (0.8)	36.3	17 (2.4)
G5 S. Span	span	interior	WA bottom	-3.5 (-0.5)	-21.9	3 (0.5)	14.1	7 (1)

The response history and temperature versus stress data (bottom flange) at each cross-section in Table 7 is presented in Figure 40 and Figure 41, respectively.

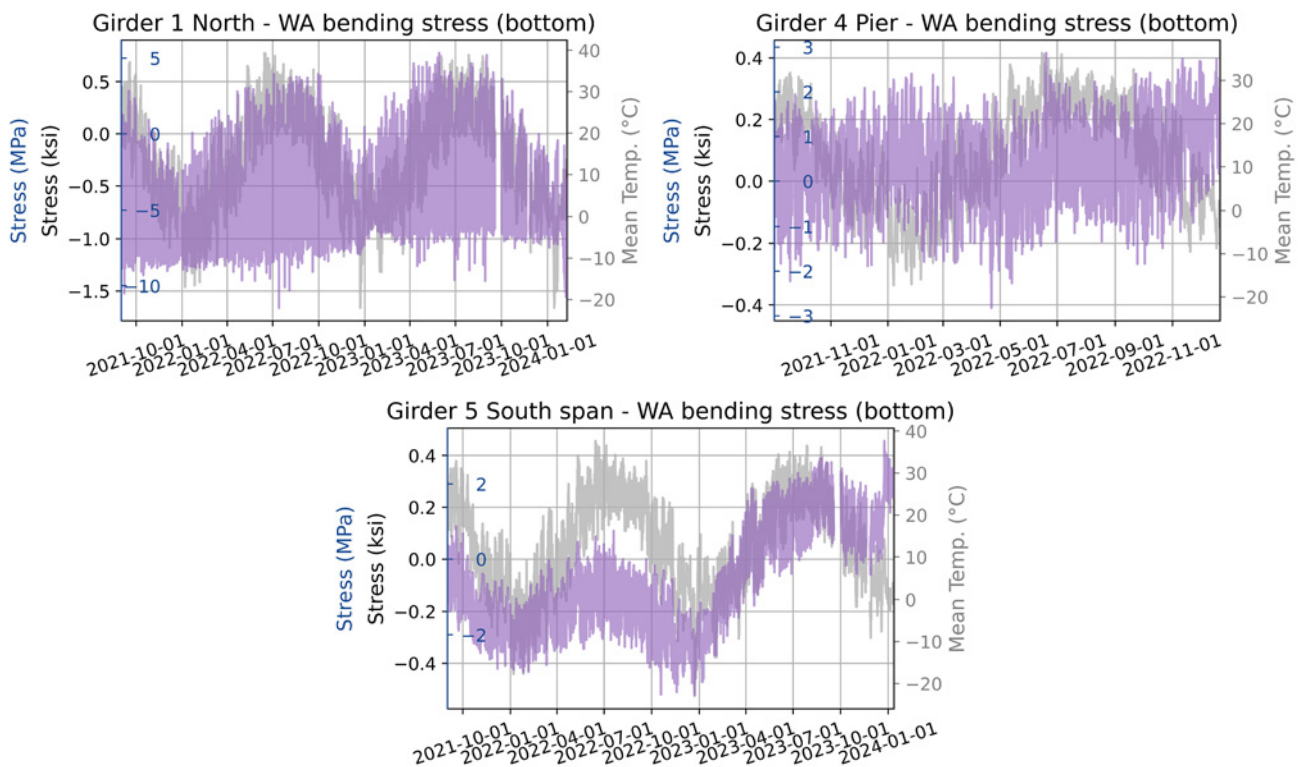


Figure 40. Graph. Weak-axis bending stress data—bottom flange.

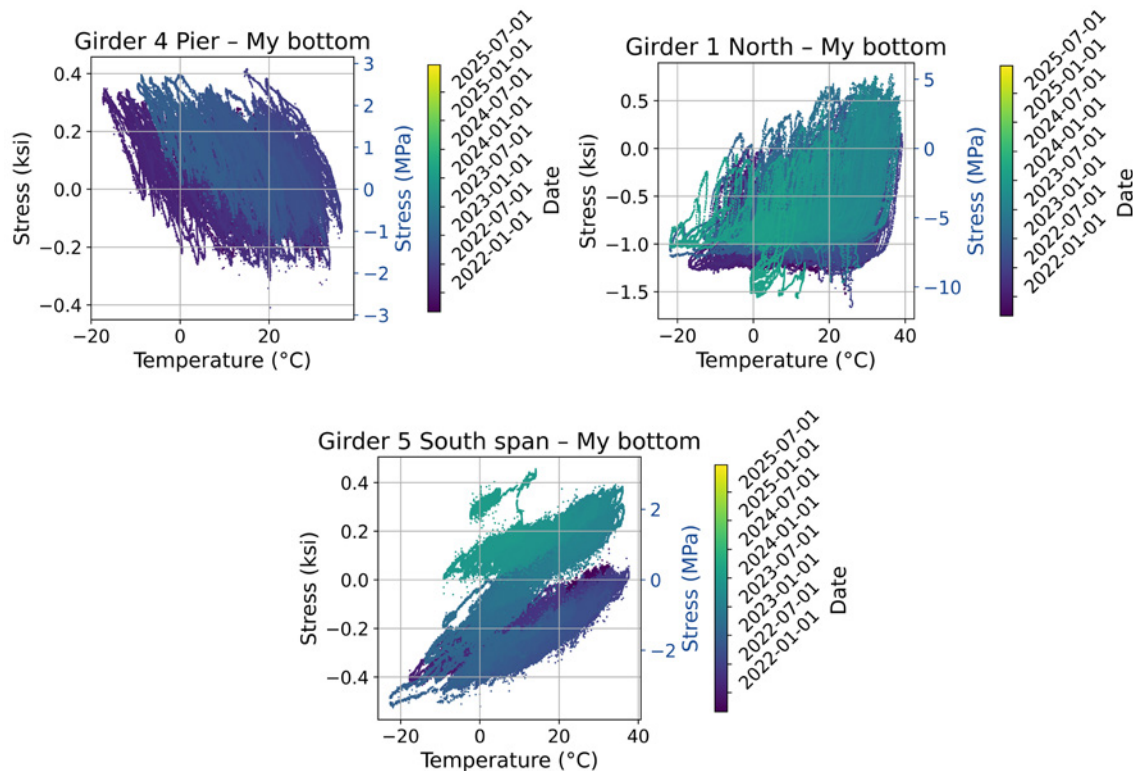


Figure 41. Graph. Temperature vs. weak-axis bending stress data—bottom flange.

The weak-axis bending response for Girder 1 North shown in the response history in Figure 40 and in the temperature versus stress relationship in Figure 41 exhibits noticeable asymmetry, with significantly larger stress variations developing during increasing temperatures than during decreasing temperatures. At this exterior girder span location, a total weak-axis bending stress range of approximately 17 MPa (2.4 ksi) was recorded over the monitoring period. Unlike locations where a more symmetric response is observed and the stress range may be interpreted as approximately equally distributed between positive and negative bending, the behavior observed at this location suggests that a significantly larger portion of the recorded range may develop in a single bending direction. This behavior may indicate that lateral bending stresses that developed solely due to thermal variations can exceed the AASHTO (2024) suggested lateral bending stress limit of 14 MPa (2 ksi) for exterior girders near midspan, even though this recommended value is intended to account for the combined effects of all unfactored loading conditions.

Mattis-57 (IAB)

Table 8 summarizes maximum and minimum weak-axis (lateral) bending measures from Mattis-57. The results are separated per location along the bridge length (pier, span, or abutment for Mattis-57) and per girder type (exterior or interior). As for other response measures, higher lateral bending is observed on the monitored IAB than on the stub abutment bridge, with a maximum stress range value of 30 MPa (4.3 ksi) at a cross-section near the abutment and a maximum of 18 MPa (2.5 ksi) near midspan, both on the exterior girder. The overall stress ranges presented herein encompass stresses developed due to both uniform temperature changes and thermal gradients.

Table 8. Summary of Bottom-Flange Weak-Axis Bending Stress Data from Girder Cross-Section Gauges—Mattis-57

Cross-Section	Location	Girder Type	Component	Min. Stress MPa (ksi)	Temp. at Min. (°C)	Max. Stress MPa (ksi)	Temp. at Max. (°C)	Stress Range MPa (ksi)
G8 N. Abut.	abut.	exterior	WA bottom	-19 (-2.8)	30.5	10.5 (1.5)	20.2	30 (4.3)
G7 S. Abut.	abut.	interior	WA bottom	-3.5 (-0.5)	33.7	4.5 (0.6)	-12.2	8 (1.2)
G6 Pier S.	pier	interior	WA bottom	-2 (-0.3)	4.2	1 (0.1)	20.2	3 (0.4)
G8 S. Span	span	exterior	WA bottom	-15 (-2.2)	29.0	2.5 (0.3)	23.8	18 (2.5)
G5 S. Span	span	interior	WA bottom	-6 (-0.9)	-7.6	0.5 (0.1)	32.6	6.5 (1)

The response history and temperature versus stress data (bottom flange) at each cross-section in Table 8 is presented in Figure 42 and Figure 43, respectively.

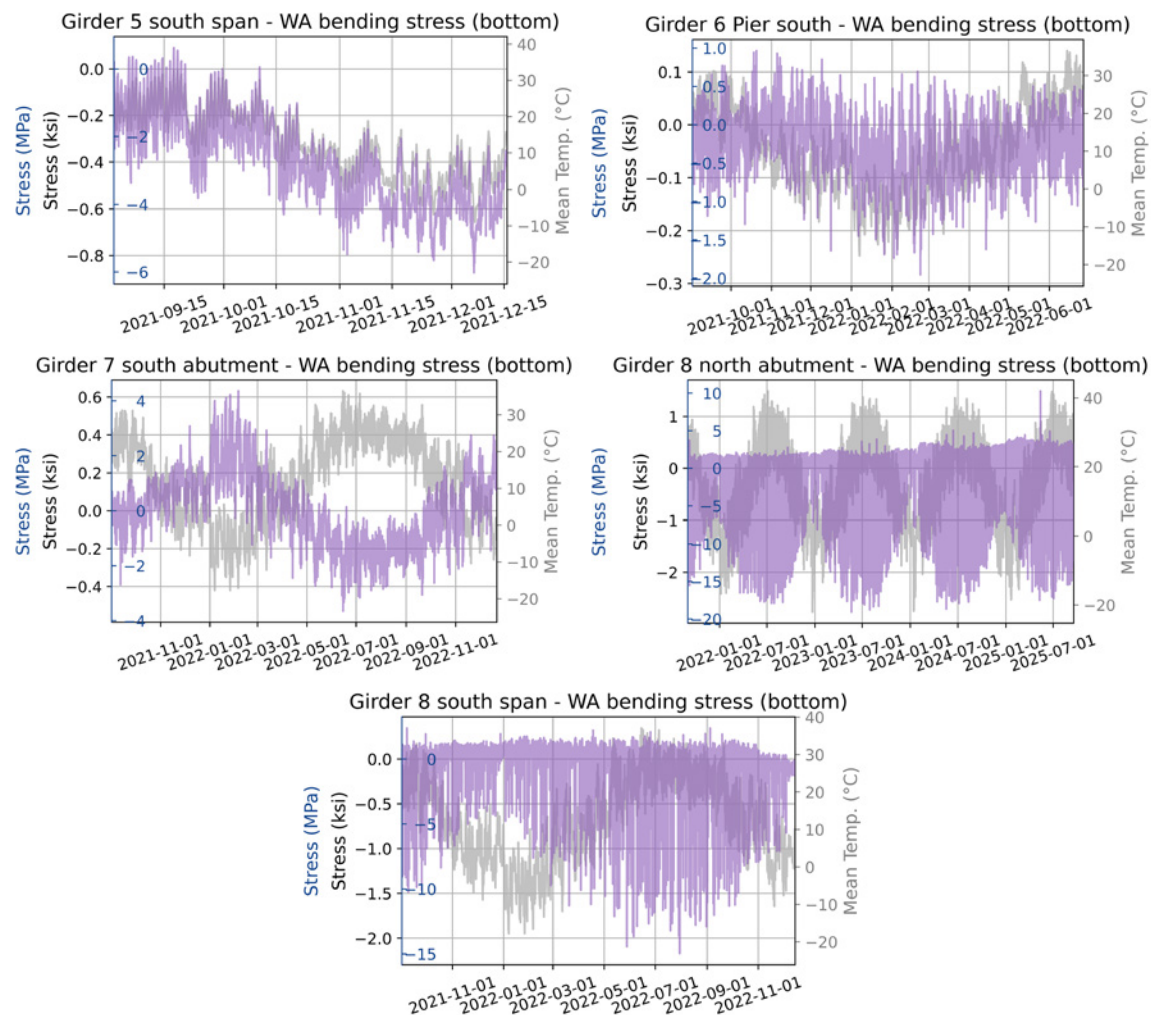


Figure 42. Graph. Weak-axis bending stress data—bottom flange.

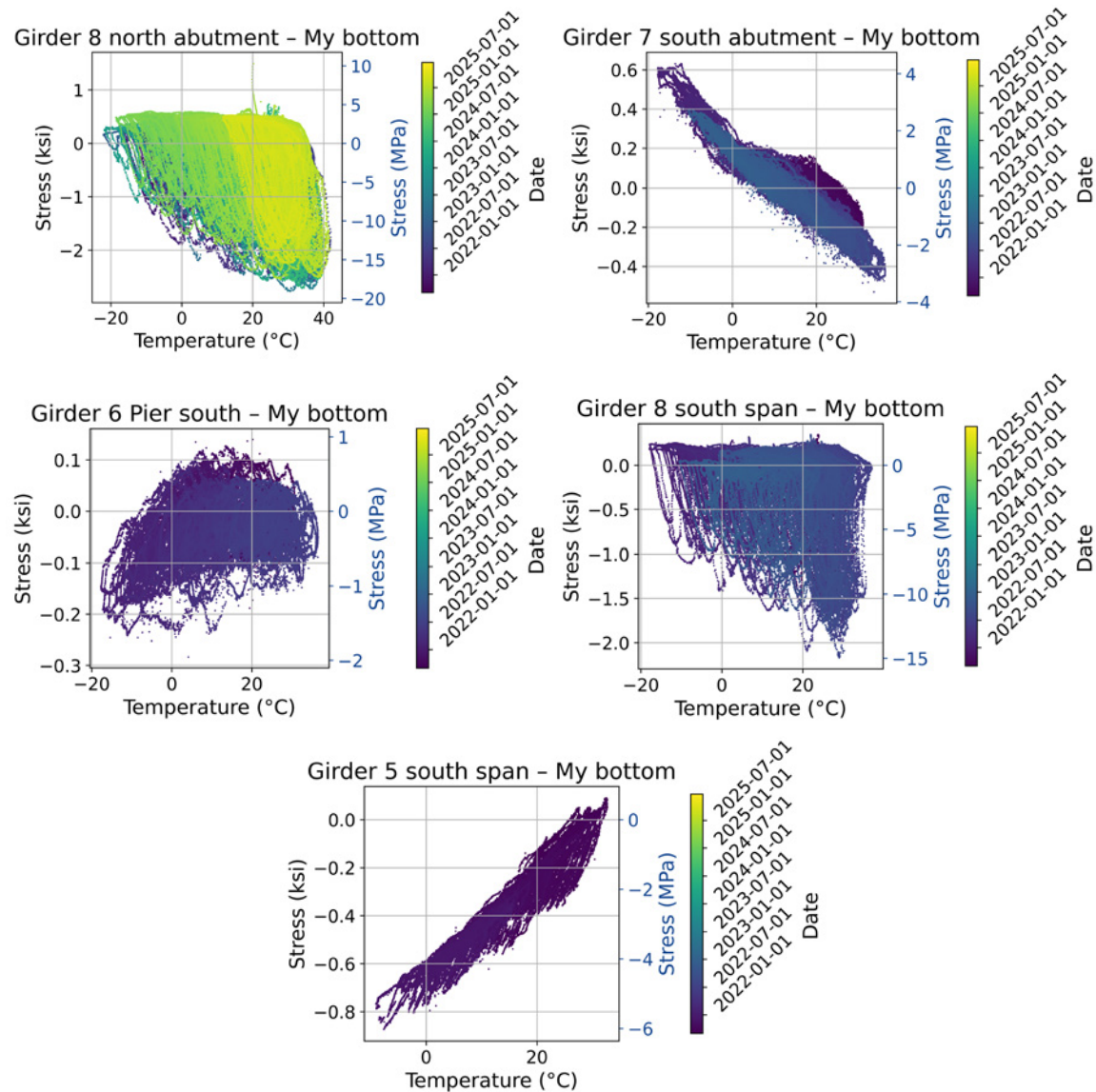


Figure 43. Graph. Temperature vs. weak-axis bending stress data—bottom flange.

These results indicate that thermal variations alone can contribute a substantial portion of the AASHTO-suggested lateral bending stress limits for exterior girders, reaching values exceeding the 14 MPa (2 ksi) recommendation near midspan and approaching the 52 MPa (7.5 ksi) recommendation near supports, even though these suggested limits are intended to account for the combined effects of all unfactored loading conditions. Furthermore, the temperature versus stress relationships for Girder 8 North Abutment and Girder 8 South Span (Figure 43) suggest that a significant portion of this lateral bending response may be associated with short-term thermal gradient effects, as the corresponding long-term temperature-stress slopes appear relatively small compared to the overall observed stress variation. Quantification of these deviations from long-term trends is presented later in this chapter.

I-GIRDER AXIAL STRESS

This section presents the axial stress component, which is directly influenced by thermal expansion and restraint conditions at the supports. The results provide insight into the development of axial forces under long-term temperature variations and their dependence on abutment configuration. Selected plots are used to illustrate maximum responses. The full set of axial stress data is presented in Appendix A.

Mattis-74 (Stub)

Table 9 summarizes maximum and minimum axial stress measures collected from Mattis-74. The results are separated per location along the bridge length (pier or span for Mattis-74) and per girder type (exterior or interior). Significant axial stress ranges (up to 32 MPa [4.6 ksi]) are recorded even for Mattis-74, which has elastomeric bearings at the end supports and fixed bearings at the pier. The overall stress ranges presented herein encompass stresses developed due to both uniform temperature changes and thermal gradients.

Table 9. Summary of Axial Stress Data from Girder Cross-Section Gauges—Mattis-74

Cross-Section	Location	Girder Type	Component	Min. Stress MPa (ksi)	Temp. at Min. (°C)	Max. Stress MPa (ksi)	Temp. at Max. (°C)	Stress Range MPa (ksi)
G4 Pier	pier	interior	Axial	−6.5 (−0.9)	25.4	10 (1.5)	−11.8	16.5 (2.4)
G1 N. Span	span	exterior	Axial	−5 (−0.7)	18.0	22 (3.2)	−14.1	27 (3.9)
G4 N. Span	span	interior	Axial	−9 (−1.3)	27.8	23 (3.3)	−22.7	32 (4.6)

The response history and temperature versus stress data at each cross-section in Table 9 is presented in Figure 44 and Figure 45, respectively.

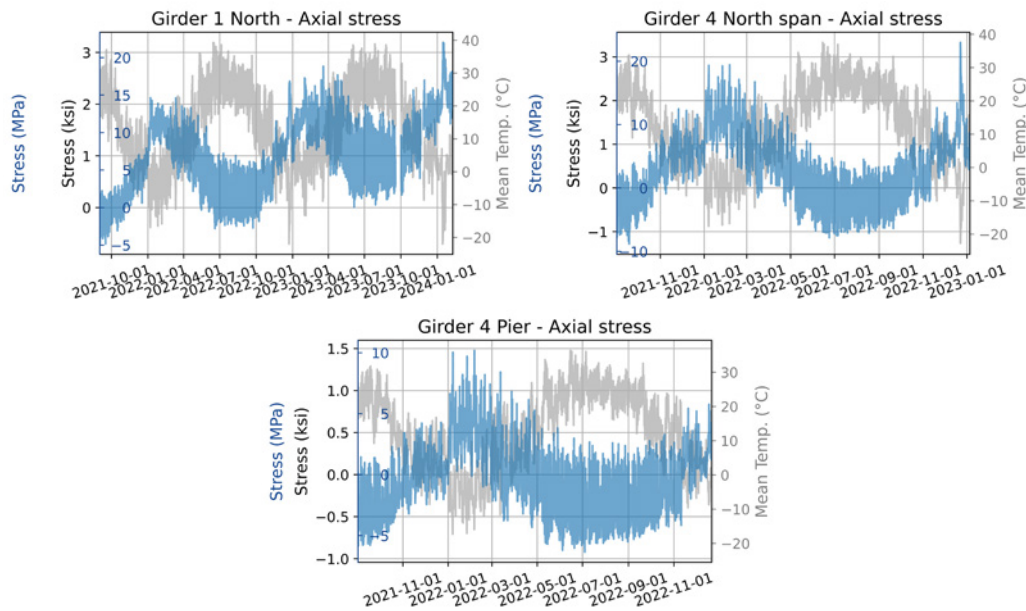


Figure 44. Graph. Axial stress data.

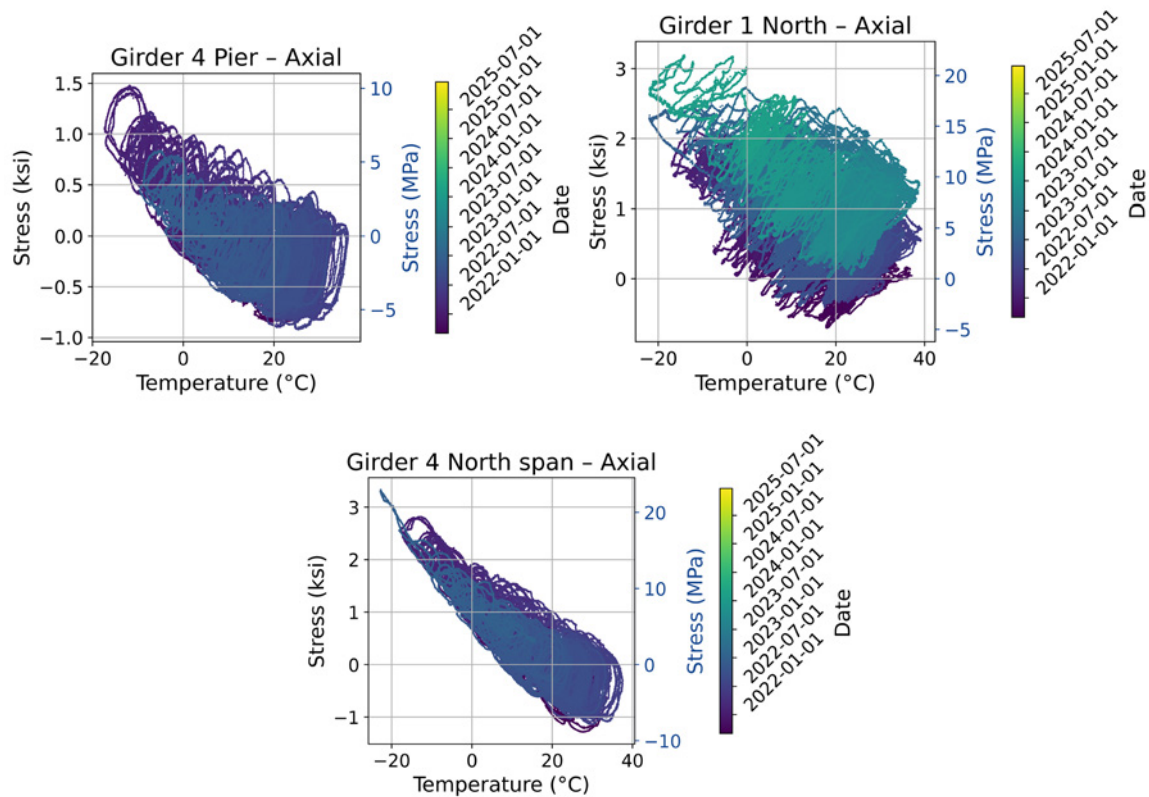


Figure 45. Graph. Temperature vs. axial stress data.

Mattis-57 (IAB)

Table 10 summarizes maximum and minimum axial stress measures collected from Mattis-57. The results are separated per location along the bridge length (pier, span, or abutment for Mattis-57) and per girder type (exterior or interior). The maximum axial stress range recorded is 39 MPa (5.6 ksi) on an exterior girder. The overall stress ranges presented herein encompass stresses developed due to both uniform temperature changes and thermal gradients.

Table 10. Summary of Axial Stress Data from Girder Cross-Section Gauges—Mattis-57

Cross-Section	Location	Girder Type	Component	Min. Stress MPa (ksi)	Temp. at Min. (°C)	Max. Stress MPa (ksi)	Temp. at Max. (°C)	Stress Range MPa (ksi)
G8 N. Abut.	abut.	exterior	Axial	-13 (-1.9)	41.5	25.5 (3.7)	-21.8	38.5 (5.6)
G6 S. Abut.	abut.	interior	Axial	-12.5 (-1.8)	34.7	23.5 (3.4)	-22.8	36 (5.2)
G6 Pier S.	pier	interior	Axial	-8 (-1.2)	33.5	12.5 (1.8)	-14.1	20.5 (2.9)
G8 S. Span	span	exterior	Axial	-10 (-1.4)	35.3	21.5 (3.1)	-17.3	31.5 (4.5)
G6 S. Span	span	interior	Axial	-10.5 (-1.5)	34.7	22.5 (3.3)	-17.4	33 (4.8)

The response history and temperature versus stress data at each cross-section in Table 10 is presented in Figure 46 and Figure 47, respectively.

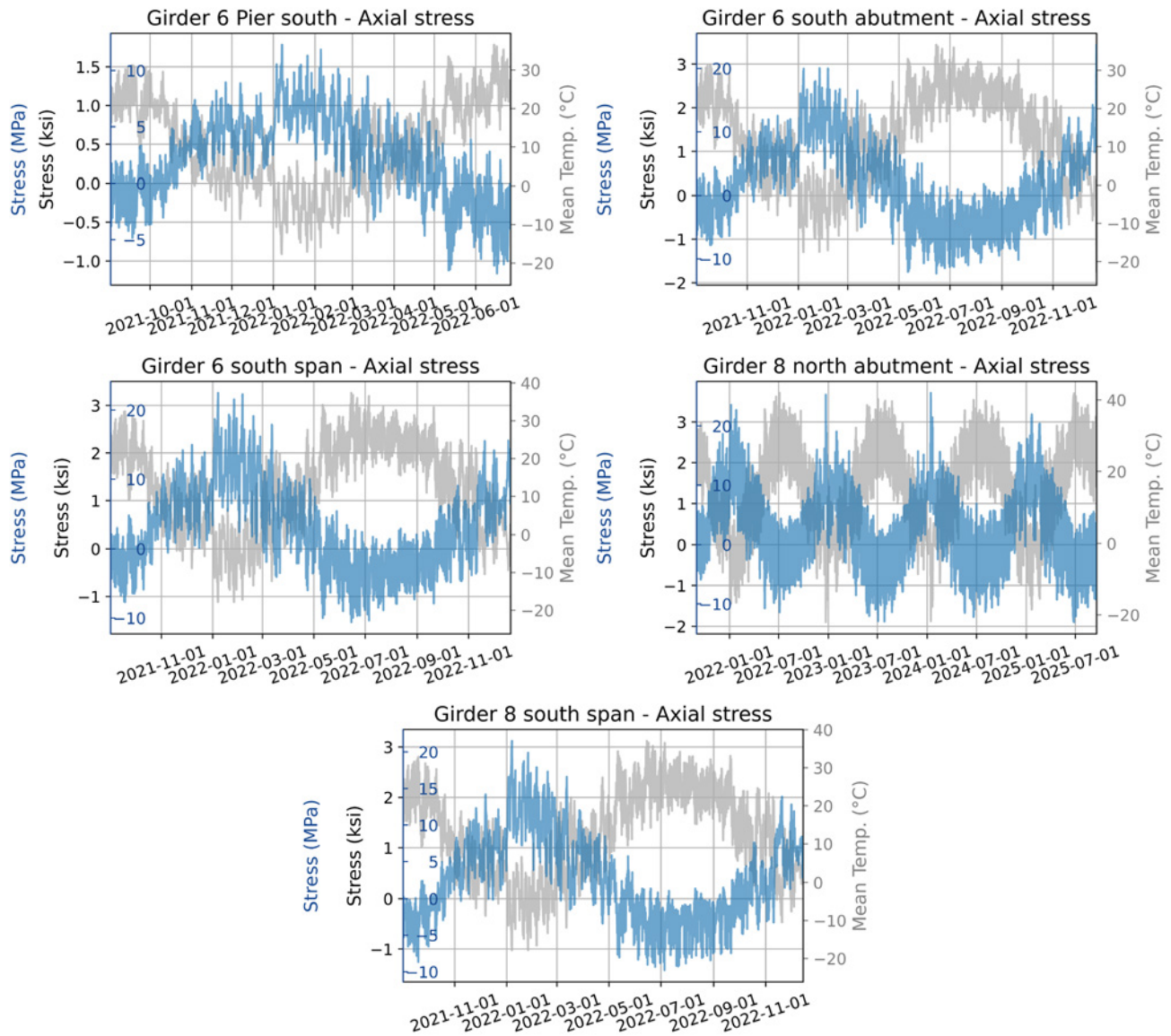


Figure 46. Graph. Axial stress data.

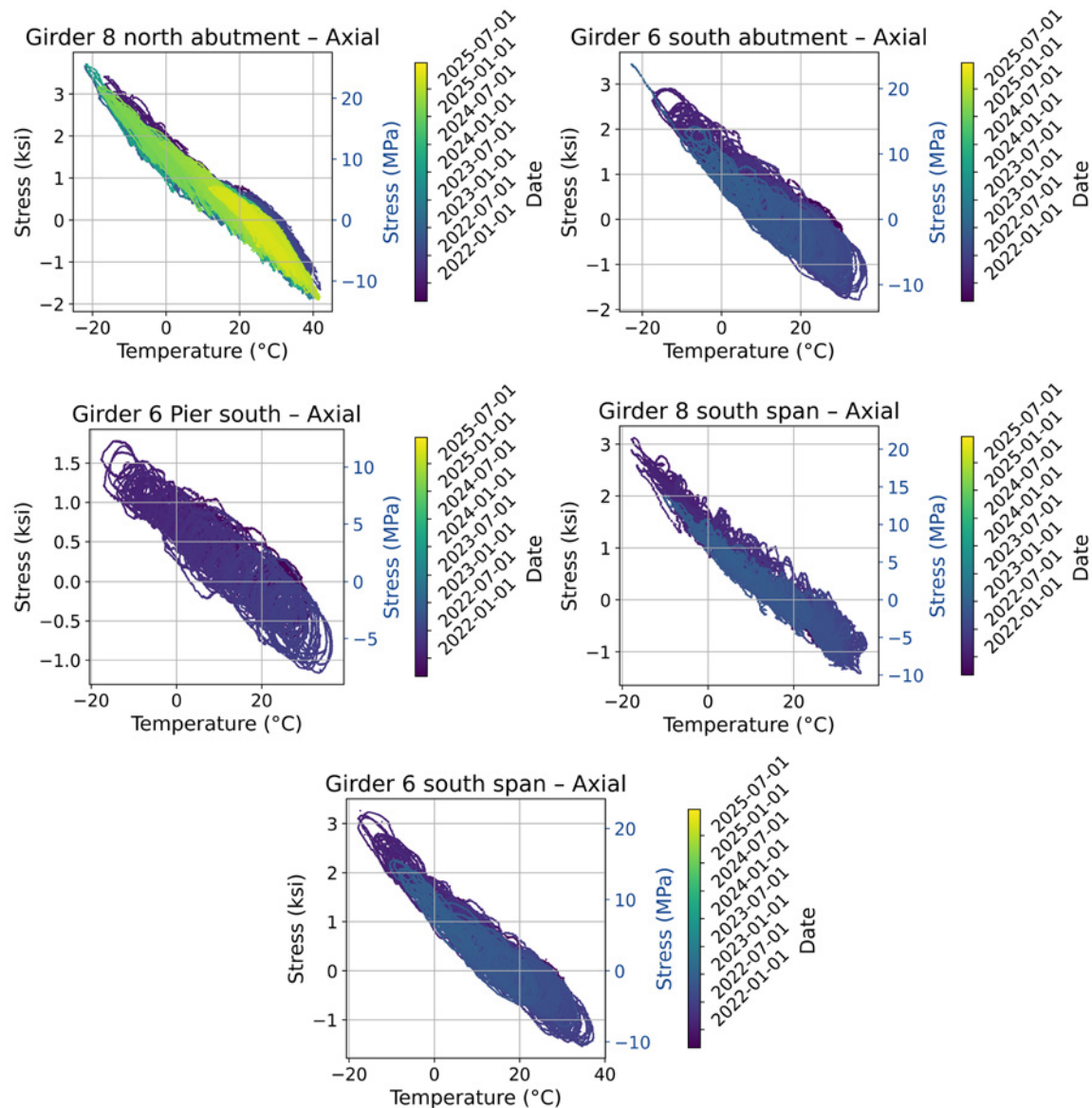


Figure 47. Graph. Temperature vs. axial stress data.

As opposed to the strong- and weak-axis bending responses, the temperature versus stress behavior observed in Figure 47 indicates that the axial stress response is dominated primarily by the long-term thermal behavior of the bridge. Short-term deviations from the dominant temperature-stress trend are comparatively small, suggesting thermal gradients have a significantly smaller influence on axial response than on bending-related stress components.

THERMAL GRADIENT EFFECTS ANALYSIS

To quantify the primary long-term structural response, a linear regression is fitted to the scatter temperature versus stress data to establish the thermal behavior and extract a baseline thermal stress coefficient (slope of the temperature versus decomposed stress plots). Although the overall

stress variation inherently includes the combined effects of both uniform temperature changes and short-term thermal gradients, the following analysis seeks to isolate the contribution of the thermal gradients by quantifying the deviation (scatter) of the measured stresses from the long-term thermal trend. The magnitude of these effects and the deviation from the long-term thermal response are evaluated using the mean absolute error (MAE) and maximum extreme error bounds, which are summarized for all cross-sections in Table 11 and Table 12 for Mattis-74 and Mattis-57, respectively. This statistical evaluation characterizes the short-term structural response of the monitored bridges, which in many cases exhibits behavior distinct from the long-term thermal trend and is most likely driven by complex thermal gradients. In addition, defining the magnitude and envelope of these short-term effects provides an important reference for evaluating the parametric finite element analyses presented in Chapter 5, since the numerical models primarily capture long-term thermal behavior and do not explicitly simulate the short-term thermal gradients observed in the field-collected temperature data. To better visualize the information in Table 11 and Table 12, Figure 48 and Figure 49 present the typical temperature versus stress responses together with their corresponding regression analyses for the axial and bottom-flange lateral bending components (at a single cross-section), respectively, for both Mattis-74 and Mattis-57.

Table 11. Summary of Linear Regression Statistics for Decomposed Thermal Stresses—Mattis-74

Cross-Section	Location	Component	Regression Slope MPa/°C (ksi/°C)	Mean Absolute Error MPa (ksi)	Max Error Range MPa (ksi)
G5 Pier	pier	Axial	−0.143 (−0.021)	1.81 (0.26)	11.80 (1.71)
G5 Pier	pier	Mx bottom	−0.116 (−0.017)	1.89 (0.27)	19.12 (2.77)
G5 Pier	pier	My bottom	−0.028 (−0.004)	0.38 (0.06)	3.20 (0.46)
G4 Pier	pier	Axial	−0.169 (−0.024)	1.68 (0.24)	11.04 (1.60)
G4 Pier	pier	Mx bottom	−0.065 (−0.009)	1.86 (0.27)	18.60 (2.70)
G4 Pier	pier	My bottom	−0.029 (−0.004)	0.61 (0.09)	5.54 (0.80)
G4 N. Span	span	Axial	−0.405 (−0.059)	2.10 (0.30)	14.91 (2.16)
G4 N. Span	span	Mx bottom	0.197 (0.029)	2.36 (0.34)	24.90 (3.61)
G4 N. Span	span	My bottom	0.007 (0.001)	0.35 (0.05)	3.51 (0.51)
G1 N. Span	span	Axial	−0.214 (−0.031)	3.17 (0.46)	23.04 (3.34)
G1 N. Span	span	Mx bottom	−0.016 (−0.002)	3.79 (0.55)	42.50 (6.16)
G1 N. Span	span	My bottom	0.076 (0.011)	1.37 (0.20)	16.48 (2.39)
G5 S. Span	span	Axial	−0.342 (−0.050)	2.06 (0.30)	14.52 (2.11)
G5 S. Span	span	Mx bottom	0.161 (0.023)	2.67 (0.39)	27.28 (3.96)
G5 S. Span	span	My bottom	0.061 (0.009)	0.98 (0.14)	6.25 (0.91)

Table 12. Summary of Linear Regression Statistics for Decomposed Thermal Stresses—Mattis-57

Cross-Section	Location	Component	Regression Slope MPa/°C (ksi/°C)	Mean Absolute Error MPa (ksi)	Max Error Range MPa (ksi)
G6 Pier S.	pier	Axial	−0.290 (−0.042)	1.29 (0.19)	9.58 (1.39)
G6 Pier S.	pier	Mx bottom	0.089 (0.013)	2.32 (0.34)	18.70 (2.71)
G6 Pier S.	pier	My bottom	0.011 (0.002)	0.27 (0.04)	2.87 (0.42)
G8 N. Abut.	abut.	Axial	−0.493 (−0.072)	1.07 (0.16)	10.94 (1.59)
G8 N. Abut.	abut.	Mx bottom	−0.492 (−0.071)	4.42 (0.64)	38.09 (5.52)
G8 N. Abut.	abut.	My bottom	−0.106 (−0.015)	2.02 (0.29)	28.30 (4.10)
G5 S. Span	span	Axial	−0.467 (−0.068)	1.38 (0.20)	9.93 (1.44)
G5 S. Span	span	Mx bottom	−0.030 (−0.004)	3.13 (0.45)	25.05 (3.63)
G5 S. Span	span	My bottom	0.137 (0.020)	0.30 (0.04)	2.45 (0.36)
G6 S. Abut.	abut.	Axial	−0.518 (−0.075)	1.83 (0.27)	13.28 (1.93)
G6 S. Abut.	abut.	Mx bottom	−0.150 (−0.022)	3.78 (0.55)	29.83 (4.33)
G6 S. Abut.	abut.	My bottom	−0.067 (−0.010)	0.56 (0.08)	6.15 (0.89)
G6 S. Span	span	Axial	−0.496 (−0.072)	1.54 (0.22)	11.52 (1.67)
G6 S. Span	span	Mx bottom	−0.034 (−0.005)	2.96 (0.43)	26.79 (3.89)
G6 S. Span	span	My bottom	−0.032 (−0.005)	0.19 (0.03)	2.23 (0.32)
G7 S. Abut.	abut.	Axial	−0.422 (−0.061)	2.20 (0.32)	17.97 (2.61)
G7 S. Abut.	abut.	Mx bottom	−0.231 (−0.034)	4.66 (0.68)	37.50 (5.44)
G7 S. abut.	abut.	My bottom	−0.107 (−0.015)	0.32 (0.05)	3.70 (0.54)
G8 S. abut.	abut.	Axial	−0.381 (−0.055)	1.92 (0.28)	16.23 (2.35)
G8 S. abut.	abut.	Mx bottom	−0.187 (−0.027)	5.86 (0.85)	43.20 (6.27)
G8 S. abut.	abut.	My bottom	0.005 (0.001)	0.99 (0.14)	12.96 (1.88)
G8 S. span	span	Axial	−0.478 (−0.069)	1.06 (0.15)	10.33 (1.50)
G8 S. span	span	Mx bottom	−0.209 (−0.030)	3.32 (0.48)	35.94 (5.21)
G8 S. span	span	My bottom	−0.049 (−0.007)	1.33 (0.19)	17.09 (2.48)

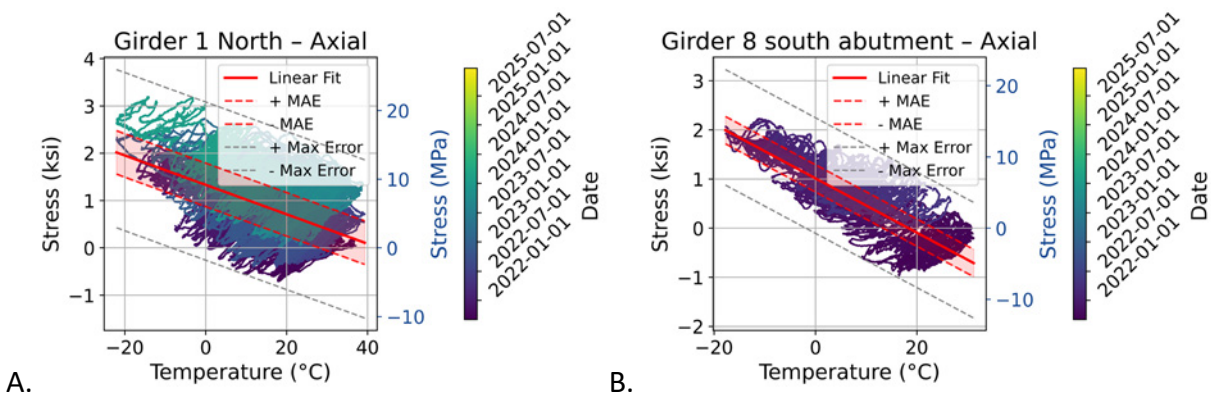


Figure 48. Graph. Typical temperature vs. axial stress response and regression analysis for (A) Mattis-74 and (B) Mattis-57.

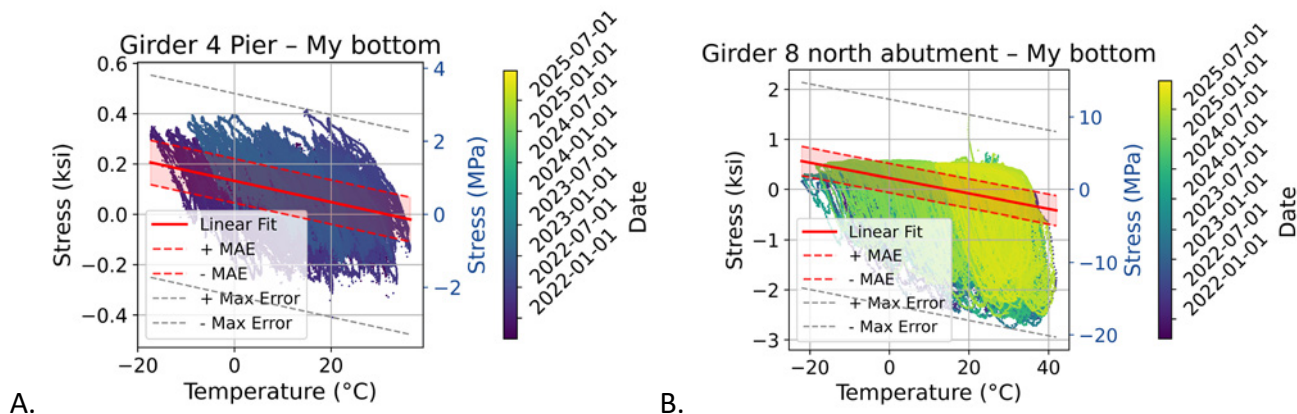


Figure 49. Graph. Typical temperature vs. bottom-flange lateral bending stress response and regression analysis for (A) Mattis-74 and (B) Mattis-57.

An evaluation of the regression dispersion shows that both bridges experience short-term axial stress variations that deviate from the primary uniform temperature response. Overall, Mattis-74 exhibits slightly higher axial dispersion—with a global average MAE of 2.1 MPa (0.31 ksi) and an average extreme error range of 15 MPa (2.2 ksi)—compared to Mattis-57 (average MAE of 1.6 MPa [0.23 ksi] and an average error range of 12.4 MPa [1.8 ksi]).

When characterizing strong-axis flexural behavior, the short-term response appears to be highly significant in both structures, though it is much more severe on the IAB. Globally, the bottom flange strong-axis bending scatter in Mattis-57 yields an average MAE of 3.8 MPa (0.55 ksi) and an average maximum error range of 32 MPa (4.7 ksi), substantially exceeding the overall scatter observed in Mattis-74 (average MAE of 2.5 MPa [0.36 ksi] and an average maximum error range of 27 MPa [3.9 ksi]), potentially indicating higher susceptibility of the IAB to thermal gradient effects. On both bridges, continuous pier locations remain with relatively small deviations from long-term trends, limiting error ranges to 19 MPa (2.8 ksi) on I-74 and 18.5 MPa (2.7 ksi) on Mattis-57. Span locations on Mattis-74 record maximum error ranges up to 42.5 MPa (6.2 ksi), while abutment and interior span regions on Mattis-57 exhibit extreme short-term variations peaking at 43 MPa (6.3 ksi) and 36 MPa (5.2 ksi), respectively. This demonstrates that vertical thermal gradients induce substantial flexural demands near span and abutment regions, generating short-term stress extremes that can be comparable or larger than those from the long-term thermal trend.

For bottom flange weak-axis bending dispersion, Mattis-74 exhibits a global average MAE of 0.75 MPa (0.11 ksi) and an average maximum error range of 7 MPa (1.0 ksi), while Mattis-57 shows similar dispersion, with an average MAE of 0.75 MPa (0.11 ksi) and an average maximum error range of 9.5 MPa (1.4 ksi). On both bridges, variation from long-term trends near the pier are smaller than at other locations. Mattis-74 span cross-sections experience lateral dispersion with extreme ranges reaching 17 MPa (2.4 ksi). On Mattis-57, lateral bending variations are more pronounced at the abutments, reaching maximum error ranges of 28 MPa (4.1 ksi), and similar to Mattis-74 within the spans, reaching 18 MPa (2.5 ksi).

These results establish a quantitative basis for evaluating the magnitude of short-term thermal effects relative to the long-term seasonal trends captured by the 3D FE models used in the parametric study presented in Chapter 5. The observed deviations demonstrate that localized short-term behavior—most likely associated with thermal gradients—can induce stress demands that add to the idealized uniform-temperature response.

CROSS-FRAME STRESS

This section presents the raw stress data directly obtained from individual strain gauges installed on cross-frames. These results provide an overall view of the measured response and serve as a reference for identifying general trends, seasonal variations, and differences between the monitored bridges. Selected plots are used to illustrate maximum responses. The full set of cross-frame raw stress data is presented in Appendix A.

Unlike the girder instrumentation discussed previously, readings exhibiting potential strain gauge malfunction were not removed from the cross-frame dataset. Consequently, some plots (and summary tables) in this section may include localized deviations or long-term drift behavior that are not necessarily representative of the actual structural response but are retained herein to provide a complete representation of the recorded measurements.

Mattis-74 (Stub)

Table 13 summarizes maximum and minimum measures from Mattis-74. Values recorded from all instrumented west and east diagonals as well as from all instrumented bottom chords are reported.

Table 13. Summary of Stress Data from Cross-Frame Gauges—Mattis-74

Cross Frame	Member	Gauge	Min. Stress MPa (ksi)	Temp. at Min. (°C)	Max. Stress MPa (ksi)	Temp. at Max. (°C)	Stress Range MPa (ksi)
CF8 3&4	B	CF_B	−15.5 (−2.3)	32.8	53.5 (7.8)	−17.9	69 (10)
CF8 4&5	E	CF_E	−24.5 (−3.6)	27.5	40.5 (5.8)	−12.4	65 (9.4)
CF12 3&4	W	CF_W	−40.5 (−5.9)	24.0	26 (3.7)	−8.0	66 (9.6)

The response history data of all gauges in each cross-frame in Table 13 are presented in Figures 50 through 52. Additionally, temperature versus stress plots for the individual gauges presented in Table 13 are shown in Figure 53.

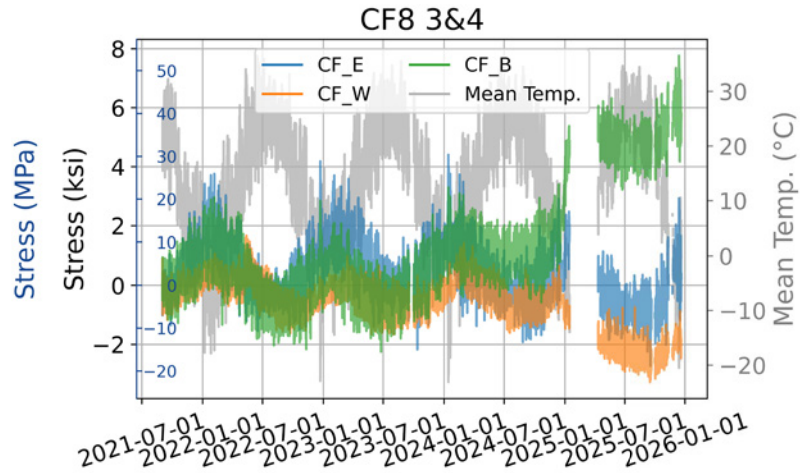


Figure 50. Graph. Stress data collected from CF8 3&4.

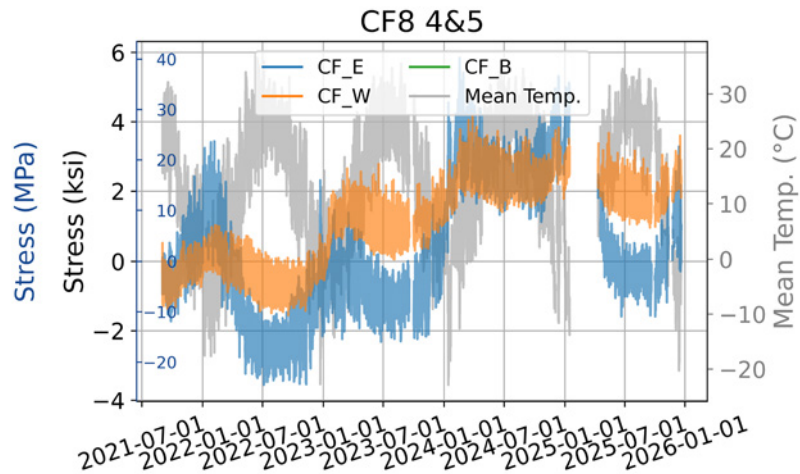


Figure 51. Graph. Stress data collected from CF8 4&5.

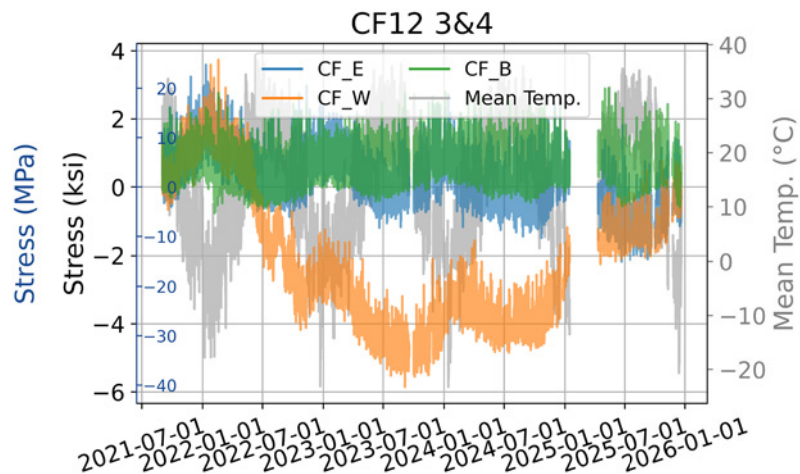


Figure 52. Graph. Stress data collected from CF12 3&4.

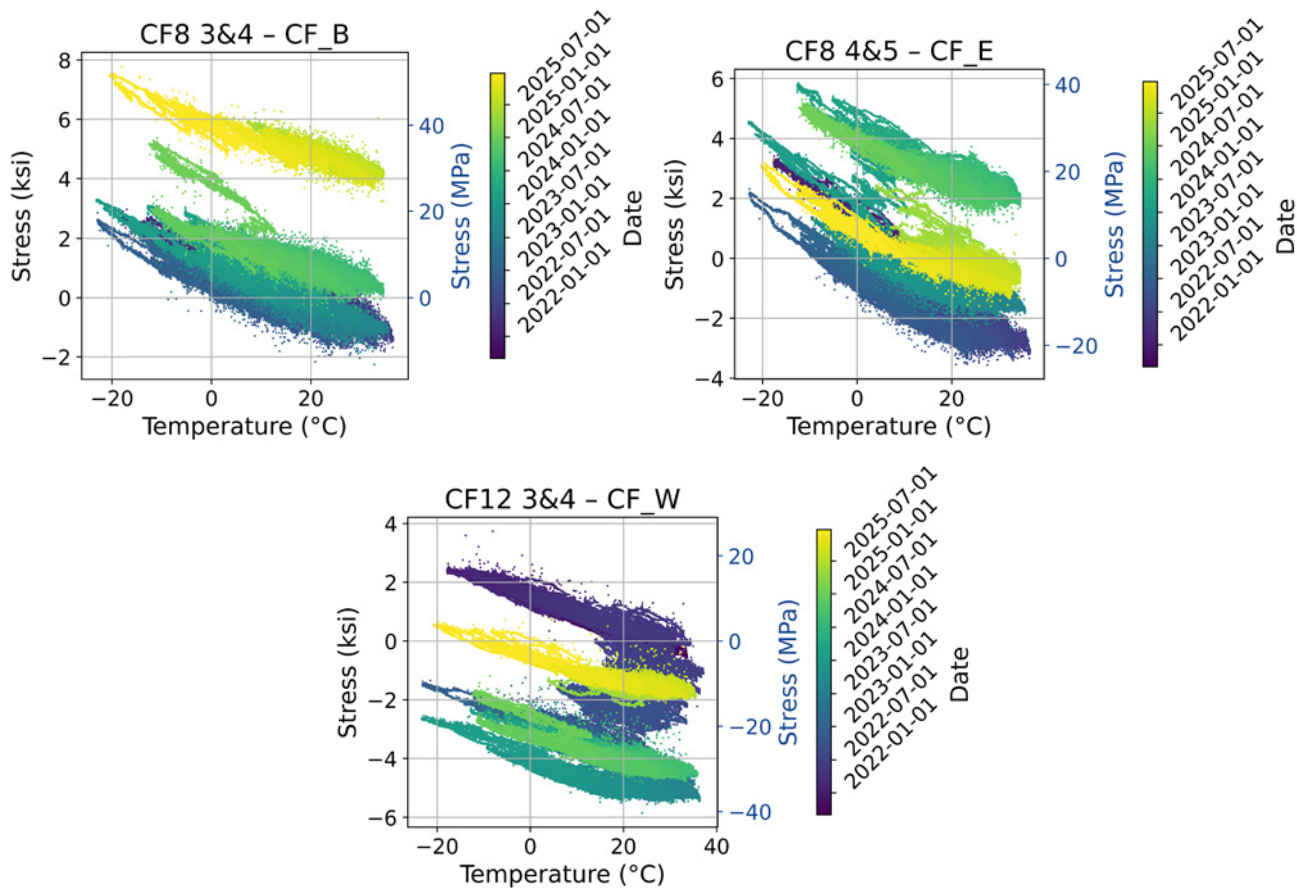


Figure 53. Graph. Temperature vs. stress data collected from different cross-frames on Mattis-74.

Mattis-57 (IAB)

Table 14 summarizes maximum and minimum strain measurements from Mattis-57. Values recorded from all instrumented west and east diagonals as well as from all instrumented bottom chords are reported.

Table 14. Summary of Stress Data from Cross-Frame Gauges—Mattis-57

Cross-Frame	Member	Gauge	Min. Stress MPa (ksi)	Temp. at Min. (°C)	Max. Stress MPa (ksi)	Temp. at Max. (°C)	Stress Range MPa (ksi)
CF1 5&6	B	CF_B	-11 (-1.6)	31.9	15.5 (2.2)	-20.0	26 (3.8)
CF17 7&8	E	CF_E	-6.5 (-0.9)	29.2	30 (4.3)	-14.3	36.5 (5.3)
CF1 5&6	W	CF_W	-20 (-2.9)	31.9	37.5 (5.4)	-20.1	57.5 (8.3)

The response history data of all gauges in each cross-frame in Table 14 are presented in Figure 54 and Figure 55. Additionally, temperature versus stress plots for the individual gauges presented in Table 14 are shown in Figure 56.

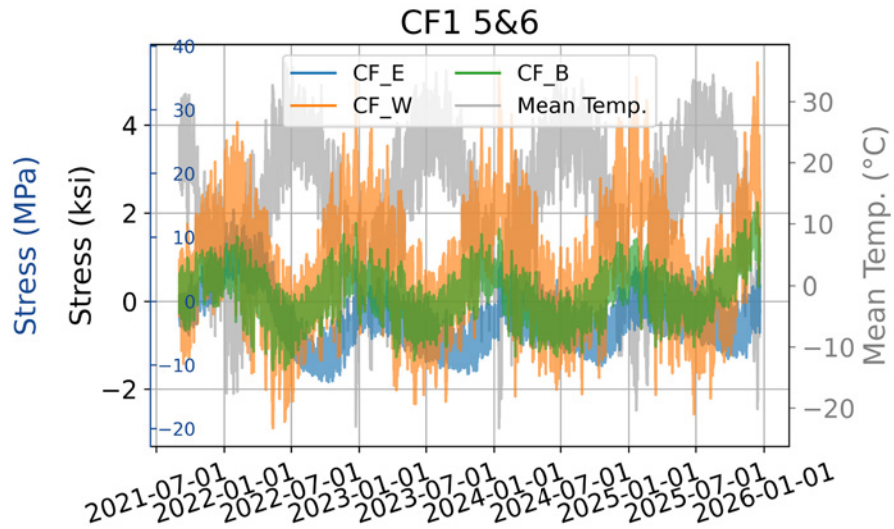


Figure 54. Graph. Stress data collected from CF1 5&6.

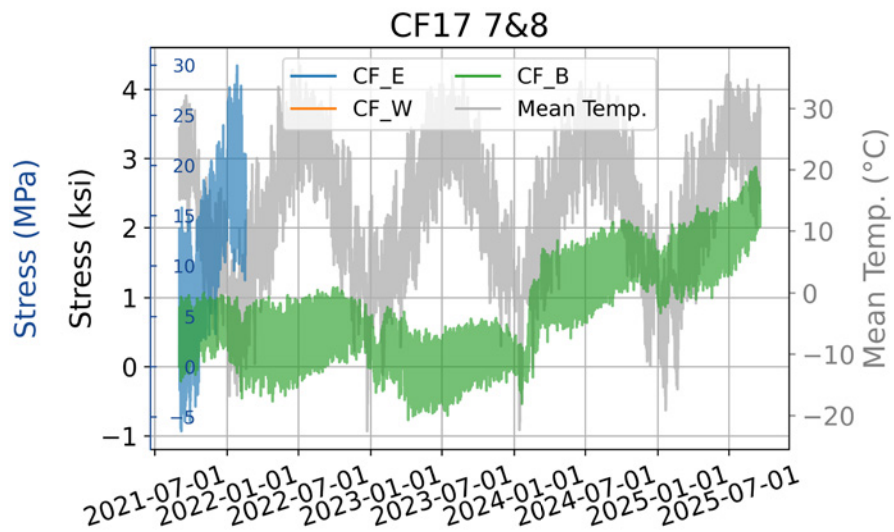


Figure 55. Graph. Stress data collected from CF17 7&8.

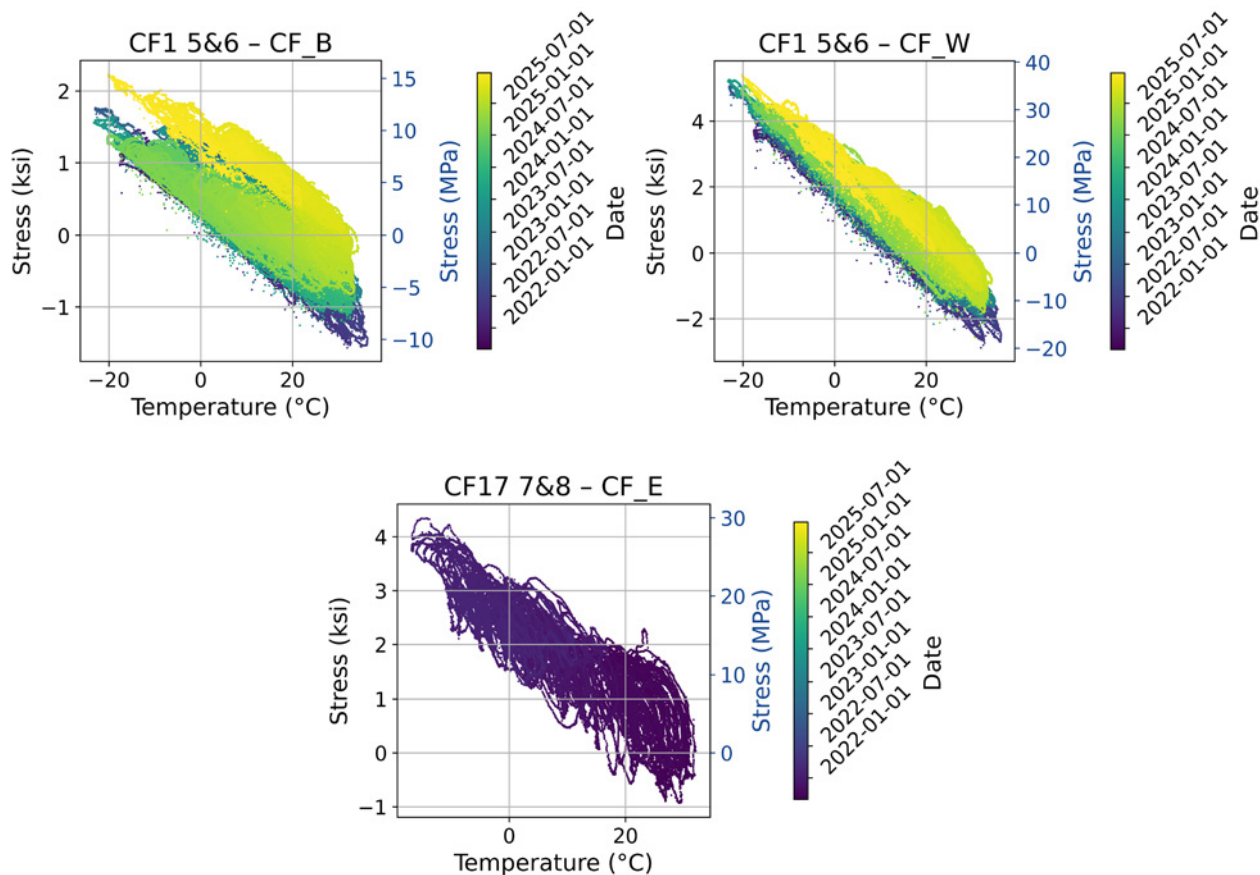


Figure 56. Graph. Temperature vs. stress data collected from different cross-frames on Mattis-57.

CRACKMETER DISPLACEMENTS

This section presents the displacement data obtained from crackmeters installed on both bridges. These results provide an overall view of the measured response and serve as a reference for identifying general trends, seasonal variations, and differences between the monitored bridges. Data are presented for all crackmeters installed. A decrease in displacement indicates compression of the crackmeter (bridge expansion), whereas an increase in displacement indicates extension of the crackmeter (bridge contraction). This expansion-contraction convention applies to longitudinal crackmeters; for transverse crackmeters, displacement behavior should be interpreted with the corresponding temperature variations.

Mattis-74 (Stub)

Table 15 summarizes the key measures for all crackmeters, including maximum, minimum, and overall range of displacement values over the monitoring period. Maximum parallel displacement ranges in the longitudinal direction reached approximately 4.2 cm (1.64 in.) at the SW corner and 4 cm (1.60 in.) at the NW corner, whereas the largest perpendicular displacement range was only about 0.8 cm (0.33 in.) at the NW corner.

Table 15. Summary of Crackmeter Displacement Data—Mattis-74

Orientation	Location	Corner Type	Min. Displ. cm (in.)	Temp. at Min. (°C)	Max. Displ. cm (in.)	Temp. at Max. (°C)	Displ. Range cm (in.)
Par.	NE	obtuse	-0.93 (-0.37)	33.8	2.22 (0.87)	-21.3	3.15 (1.24)
Par.	NW	acute	-1.13 (-0.45)	37.9	2.92 (1.15)	-22.2	4.05 (1.60)
Par.	SE	acute	-1.38 (-0.54)	35.0	1.70 (0.67)	-21.2	3.09 (1.21)
Par.	SW	obtuse	-0.47 (-0.19)	40.0	3.69 (1.45)	-20.5	4.16 (1.64)
Perp.	NE	obtuse	-0.18 (-0.07)	33.2	0.18 (0.07)	-16.3	0.36 (0.14)
Perp.	NW	acute	-0.38 (-0.15)	33.3	0.44 (0.17)	-16.6	0.83 (0.33)
Perp.	SE	acute	-0.16 (-0.06)	30.0	0.24 (0.09)	30.2	0.39 (0.15)
Perp.	SW	obtuse	-0.07 (-0.03)	34.4	0.22 (0.09)	-22.9	0.29 (0.11)

All data corresponding to Table 15 are illustrated in the response history plots presented in Figures 57 through 60, which show the full temporal evolution of the recorded displacements.

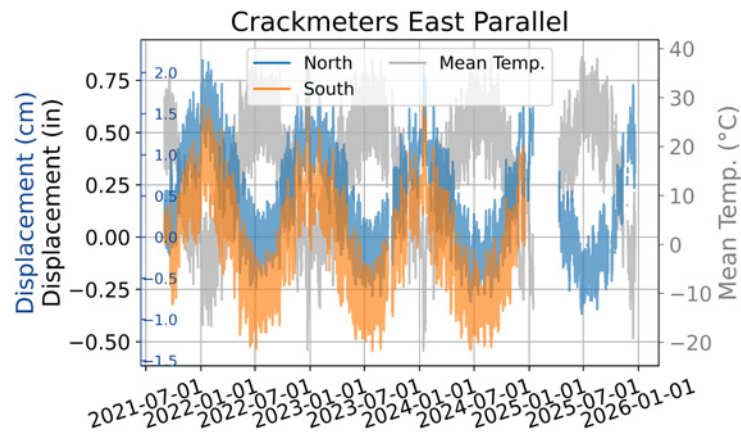


Figure 57. Graph. Displacement data collected from east crackmeters (parallel).

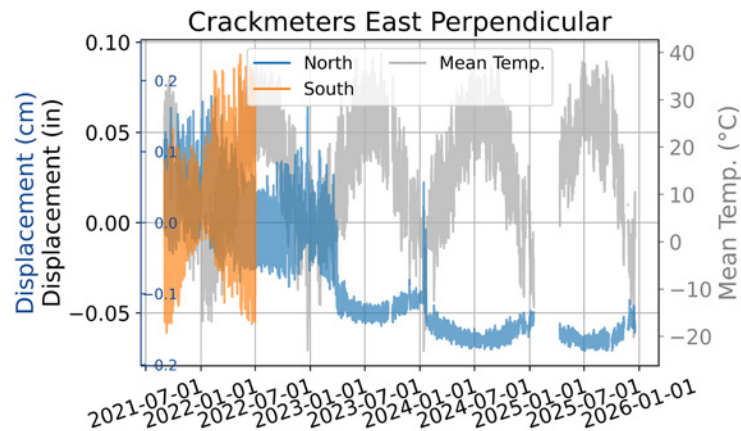


Figure 58. Graph. Displacement data collected from east crackmeters (perpendicular).

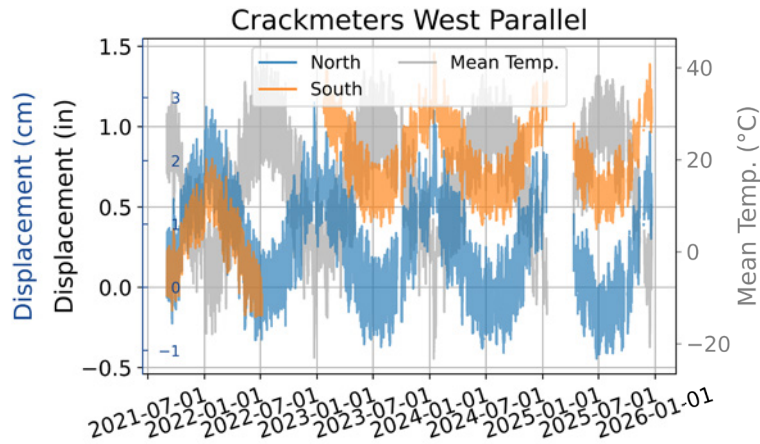


Figure 59. Graph. Displacement data collected from west crackmeters (parallel).

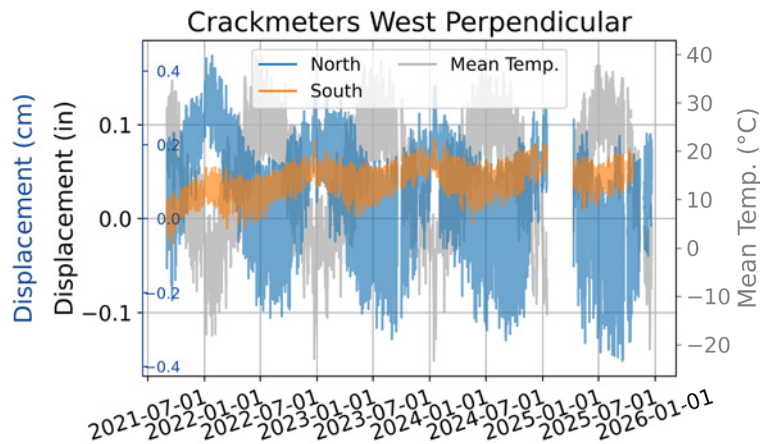


Figure 60. Graph. Displacement data collected from west crackmeters (perpendicular).

The same displacement data presented above are presented in temperature versus displacement readings in Figures 61 through 64.

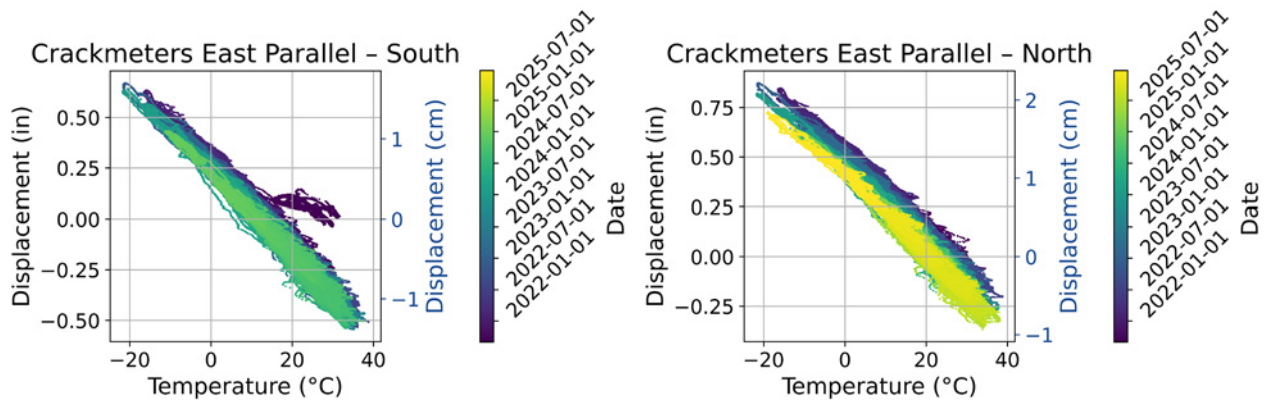


Figure 61. Graph. Temperature vs. displacement data collected from east crackmeters (parallel).

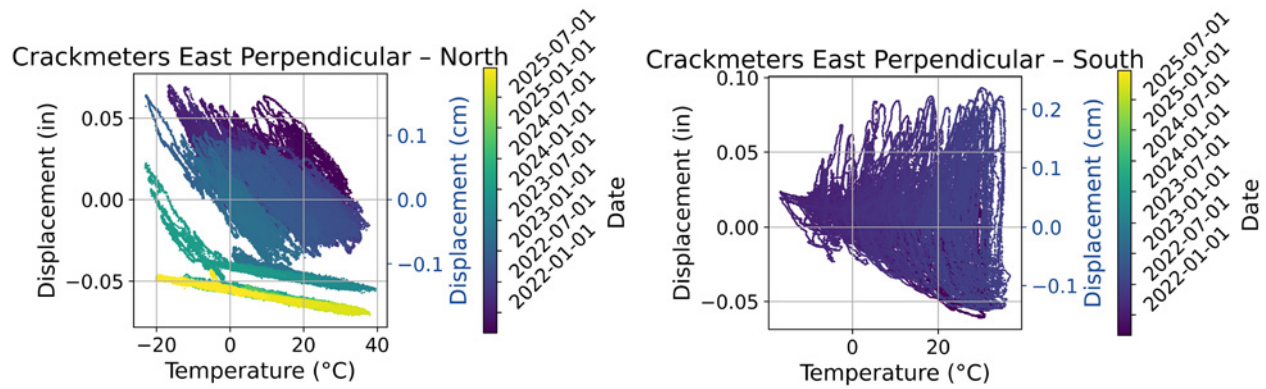


Figure 62. Graph. Temperature vs. displacement data collected from east crackmeters (perpendicular).

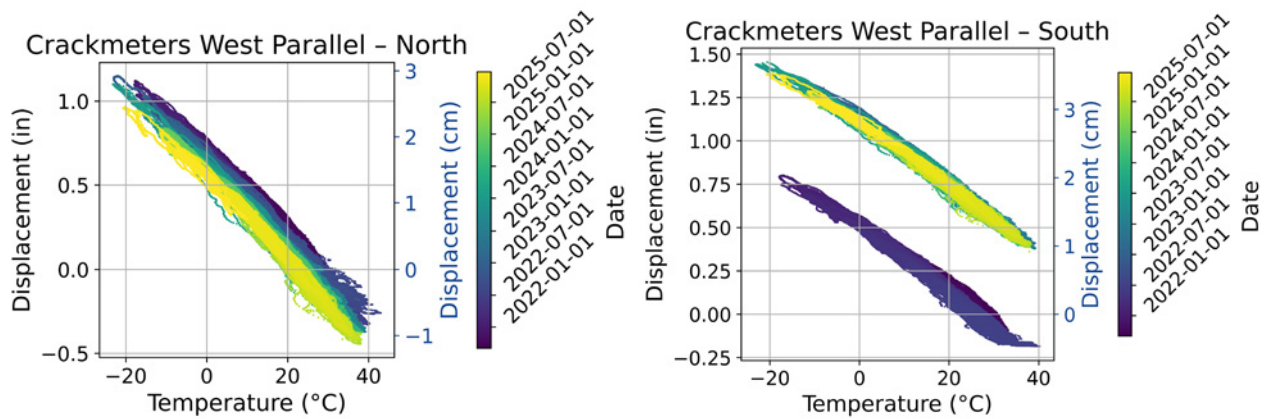


Figure 63. Graph. Temperature vs. displacement data collected from west crackmeters (parallel).

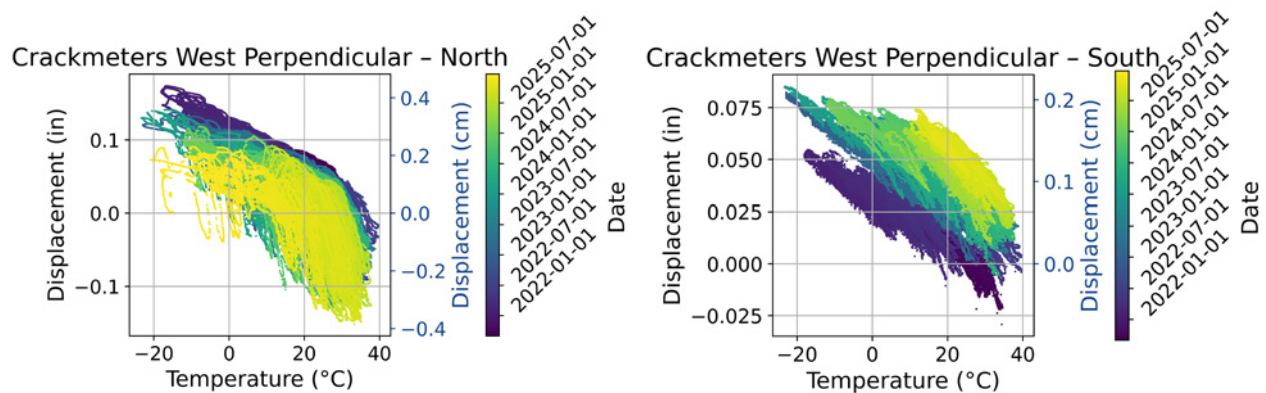


Figure 64. Graph. Temperature vs. displacement data collected from west crackmeters (perpendicular).

Mattis-57 (IAB)

Table 16 summarizes the key measures for all crackmeters, including maximum, minimum, and overall range of displacement values over the monitoring period. Longitudinal displacement ranges were relatively consistent among all monitored corners, varying from approximately 2.7 cm (1.08 in.) to 3.4 cm (1.32 in.) The largest displacement ranges were observed at the acute corners (NE and SW), while the obtuse corners (NW and SE) exhibited slightly smaller longitudinal movements.

Table 16. Summary of Crackmeter Displacement Data—Mattis-57

Orientation	Location	Corner Type	Min. Displ. cm (in.)	Temp. at Min. (°C)	Max. Displ. cm (in.)	Temp. at Max. (°C)	Displ. Range cm (in.)
Par.	NE	acute	−0.61 (−0.24)	35.6	2.67 (1.05)	−18.0	3.28 (1.29)
Par.	NW	obtuse	−0.79 (−0.31)	31.0	1.95 (0.77)	−16.8	2.74 (1.08)
Par.	SE	obtuse	−0.86 (−0.34)	36.9	1.92 (0.76)	−12.7	2.78 (1.09)
Par.	SW	acute	−0.85 (−0.34)	33.1	2.51 (0.99)	−18.1	3.36 (1.32)

All data corresponding to Table 16 are illustrated in the response history plots presented in Figure 65 and Figure 66, which show the full temporal evolution of the recorded displacements. Temperature versus stress plots for the monitored IAB are presented in Chapter 2.

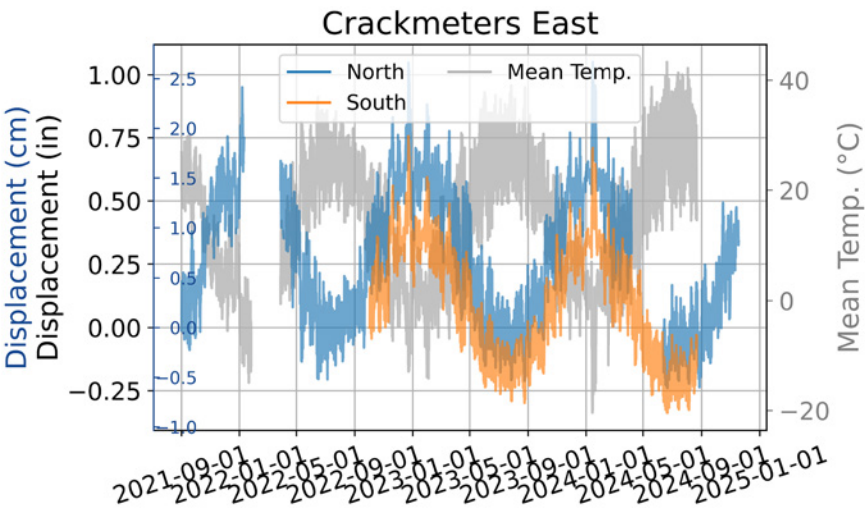


Figure 65. Graph. Displacement data collected from east crackmeters (parallel).

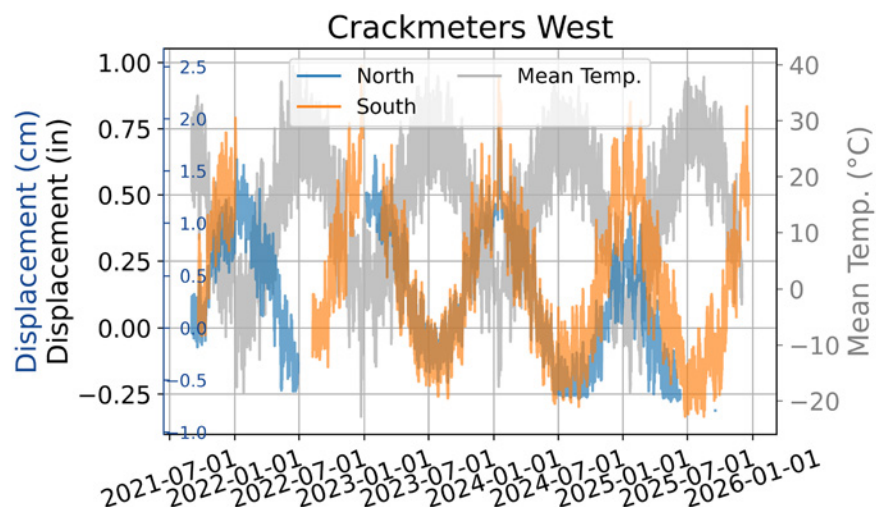


Figure 66. Graph. Displacement data collected from west crackmeters (parallel).

ABUTMENT TILT

This section presents the rotation data obtained from tiltmeters installed on both bridges. These results provide an overall view of the measured response and serve as a reference for identifying general trends, seasonal variations, and differences between the monitored bridges. Data are presented for all tiltmeters installed. Negative rotations indicate inward rotation of the girders (toward the bridge span), whereas positive rotations indicate outward rotation (away from the span).

Mattis-74 (Stub)

Table 17 summarizes the key measures for both tiltmeters, including maximum, minimum, and overall range of rotation values over the monitoring period.

Table 17. Summary of Rotation Measurements—Mattis-74

Location	Min. Rotation (deg)	Temp. at Min. (°C)	Max. Rotation (deg)	Temp. at Max. (°C)	Rotation Range (deg)
North	-0.05	30.4	0.14	-22.0	0.19
South	-0.05	30.2	0.11	-22.0	0.17

All data corresponding to Table 17 are illustrated in the response history shown in Figure 67, which shows the full temporal evolution of the recorded rotations. The same rotation data are presented in temperature versus rotation form in Figure 68.

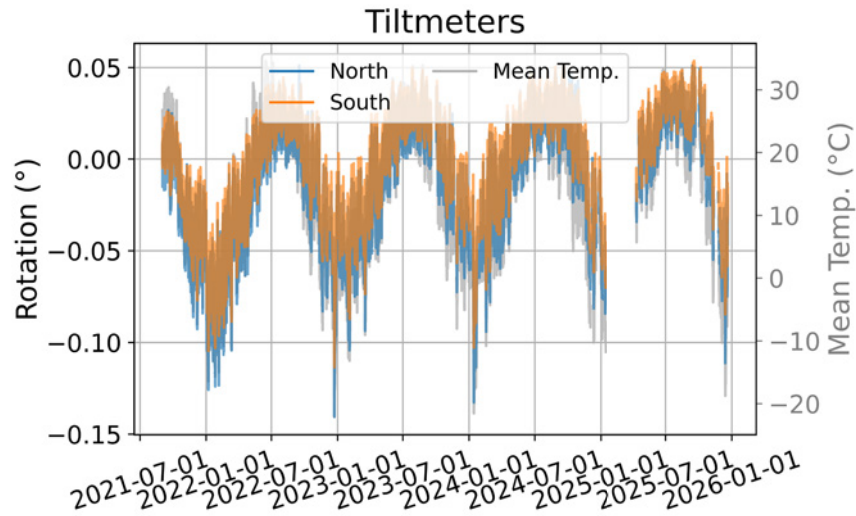


Figure 67. Graph. Rotation data collected from tiltmeters.

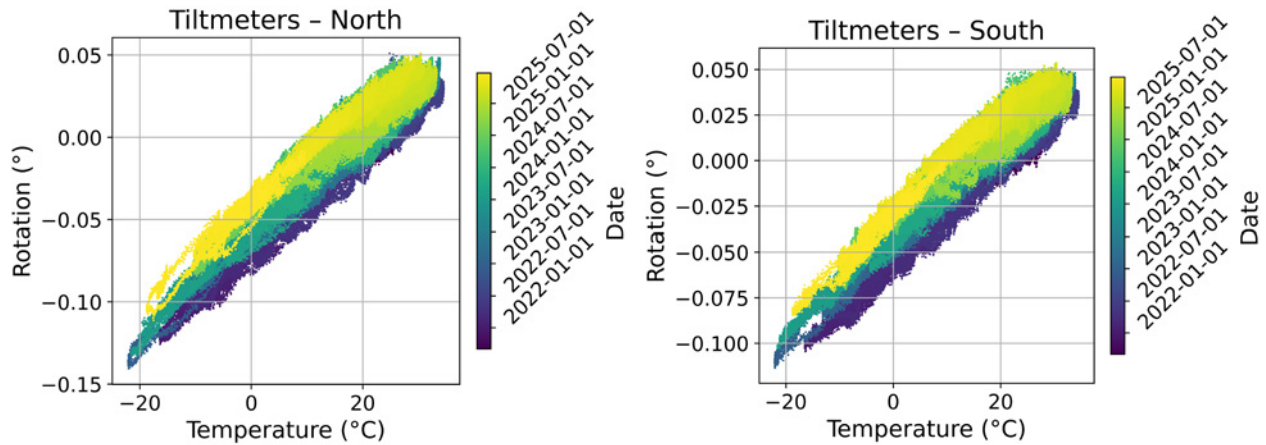


Figure 68. Graph. Temperature vs. rotation data collected from tiltmeters.

Mattis-57 (IAB)

Table 18 summarizes the key measures for both tiltmeters, including maximum, minimum, and overall range of rotation values over the monitoring period.

Table 18. Summary of Rotation Measurements—Mattis-57

Location	Min. Rotation (deg)	Temp. at Min. (°C)	Max. Rotation (deg)	Temp. at Max. (°C)	Rotation Range (deg)
North	-0.05	0.1	0.17	12.7	0.22
South	-0.27	22.3	0.13	2.4	0.40

All data corresponding to Table 18 are illustrated in the response history plots shown in Figure 69, which shows the full temporal evolution of the recorded rotations. Data in temperature versus rotation form for the monitored IAB are presented in Chapter 2.

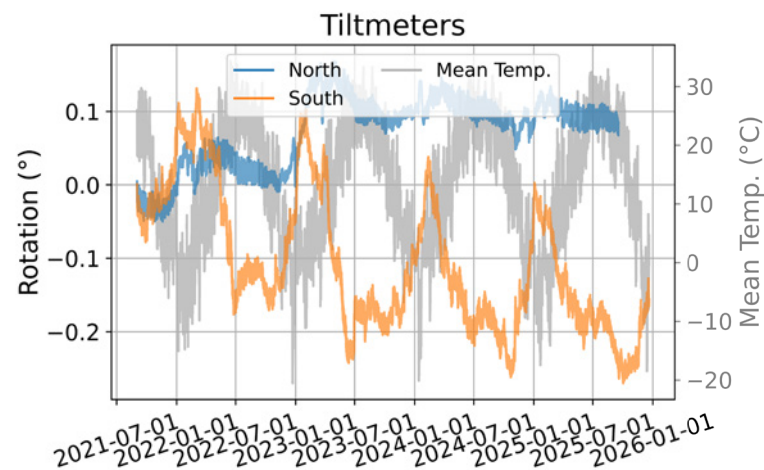
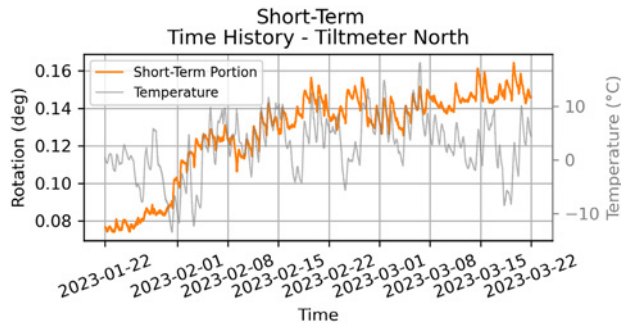


Figure 69. Graph. Rotation data collected from tiltmeters.

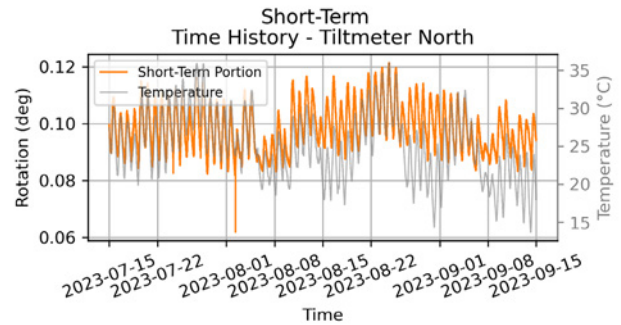
A clear temperature-rotation relationship is observed in both bridges. The correlation is noticeably stronger in the stub abutment bridge, however. The comparatively more complex relationship observed in the integral abutment bridge is likely associated with the increased interaction between the superstructure and substructure, including foundation restraint effects. Additionally, the north tiltmeter readings for the IAB appear to exhibit a more complex response and generally smaller rotation magnitudes throughout the monitoring period. For the south tiltmeter on the IAB, there seems to be a lag between the peak seasonal rotations and peak seasonal temperatures, which may be attributed to delayed thermal effects within the abutment.

Overall bridge contraction in colder months generates long-term outward abutment rotation in both tiltmeters mounted on the IAB, whereas overall bridge expansion in warmer months generates long-term inward abutment rotation. This behavior is somewhat unexpected, as the increased restraint provided by the pile foundation system would generally be expected to produce outward abutment bending during bridge expansion.

The short-term rotational response further indicates the comparatively complex behavior of the IAB. As shown in Figure 70-A, short-term rotations during colder months exhibit a weaker and less direct relationship with temperature variations. In contrast, the warmer months response shown in Figure 70-B presents a clearer correlation between short-term rotations and temperature changes, with increasing temperatures generally producing the expected outward abutment rotation associated with bridge expansion.



A. Colder periods



B. Warmer periods

Figure 70. Graph. Short-term rotation response for north tiltmeter.

The weaker correlation for the north tiltmeter might indicate more complicated behavior occurring near acute corners (as the north tiltmeter is mounted closer to an acute corner). Apart from the typical seasonal trends, an overall downward deviation is observed for the south tiltmeter readings in Figure 69, which is interpreted as an accumulation of inward rotation at the south abutment. Similarly, an overall outward deviation is observed at the north abutment.

CHAPTER 5: BRIDGE RESPONSE UNDER THERMAL LOADS—PARAMETRIC STUDY

BACKGROUND

Chapter 3 examined the influence of skewed supports, transverse load paths, and integral abutment restraint on live load–induced lateral bending in steel I-girder bridges. Skew angle, cross-frame configuration, and support restraint can significantly influence bottom-flange lateral bending stresses, particularly near supports and in bridges with integral abutments. However, while the effects of dead and live loads on lateral bending behavior have received considerable attention in previous studies (Morera 2010; Sanchez 2011; White et al. 2012, 2015, 2020; Zhou et al. 2026a), comparatively limited research has focused on the influence of thermal loads on the same response mechanisms, especially for integral abutment bridges (IABs).

Integral abutment bridges are inherently subjected to restraint forces generated by thermal expansion and contraction of the superstructure. AASHTO (2024) recognizes the importance of thermal demands and bridge movements in the design of IABs; however, current guidance related to integral bridge behavior remains limited. As summarized in Table 19, existing AASHTO provisions provide little explicit guidance regarding thermal-induced superstructure behavior, despite substantial changes in IDOT design practices over recent decades, including increases in allowable bridge length, skew angle limits, and pile flexibility considerations. Furthermore, neither AASHTO nor current IDOT provisions explicitly address lateral bending demands resulting from thermal loads, even though the same mechanisms governing lateral bending under live load—such as skewed supports, transverse load paths, and stiff support restraint—are also present under restrained thermal expansion and contraction.

Table 19. Current Design Practices Related to Integral Abutment Bridges

Parameter	IDOT Manual (2012)	IDOT Manual (2023)	AAHSTO LRFD (2024)
Max. steel IABs length	95 m (310 ft)	185 m (610 ft)	NA
Max. single-span length	NA	50 m (170 ft)	NA
Max end-span length (multi-span structure)	NA	60 m (200 ft)	NA
Max. skew for LGA	30°	45°	NA
Pile orientation	Strong axis	Weak axis	NA
Pile limit state	No yielding allowed	Moderate yielding allowed	NA
Curved girders	Not allowed	Not allowed	NA
Distance of first cross-frame from abutment	0.6 m (2 ft) from inside face of abutment	0.6 m (2 ft) from inside face of abutment	$\text{Max}(4b_f, 0.4L_b)$

The work presented in this chapter focuses on the thermal behavior of skewed steel I-girder bridges through a parametric study performed using validated 3D finite element (FE) models for bridges in the parametric study matrix presented in Chapter 2. Particular emphasis is placed on bottom-flange

lateral bending stresses and axial stresses resulting from uniform bridge temperature variations. The analyses consider uniform temperature changes ranging from +75°F to –75°F to evaluate bridge response under extreme thermal conditions representative of long-term seasonal behavior, following AASHTO (2024) temperature range for procedure “A” (Section 3.12.2.1). Unlike short-term daily thermal fluctuations, which are strongly influenced by thermal gradients, the present study focuses primarily on the dominant long-term thermal response associated with overall bridge expansion and contraction.

The FE modeling framework adopted in this study explicitly captures the influence of substructure flexibility and support restraint, including pier flexibility, bearing behavior, and soil-structure interaction at integral abutments. The modeling approach and its validation against field-monitored bridge behavior were presented in Chapter 2, where good agreement between FE-predicted and measured long-term responses was observed. Additional details regarding model development, material assumptions, boundary conditions, and validation procedures are not repeated herein and may be found in Chapter 2.

The objective of this chapter is not to propose immediate revisions to current lateral bending design provisions, particularly given that significant historical performance issues associated with thermal-induced lateral bending have not been widely reported. Rather, the goal is to better characterize and understand the behavior of skewed steel I-girder IABs subjected to restrained thermal demands, including bridge geometries both within and outside the limits commonly accepted in current design practice, and potentially support expansion of these limits and the use of IABs. Through this effort, the study seeks to identify the mechanisms governing thermally induced axial and lateral bending stress behavior, evaluate the spatial distribution and magnitude of these demands throughout the bridge system, and provide context for interpreting current design assumptions related to integral bridge behavior.

PREVIOUS STUDIES

Substantial research has been conducted in Illinois to better understand the behavior of integral abutment bridges. Initial studies by Olson et al. (2009) primarily focused on substructure response, particularly the performance of foundation piles subjected to thermally induced bridge movements. Based on these findings, several moment-relief mechanisms were proposed to reduce pile demands and extend the practical length limits of integral abutment bridges. Subsequent work by Olson et al. (2013) expanded the investigation to include the interaction between the bridge superstructure, abutments, foundation system, and surrounding soil through 3D parametric finite element analyses and field instrumentation of two Illinois bridges. The study concluded that proper detailing can effectively limit pile stresses even in highly skewed bridges and recommended that bridge design limits should be based on the combined effects of bridge length and skew rather than skew alone. These studies ultimately led to revised recommendations for maximum bridge lengths and skews for Illinois integral abutment bridges. Nevertheless, the authors noted that additional investigation of superstructure performance under extreme thermal loading would be necessary before further expanding the use of integral abutment bridges in Illinois.

To address this need, a subsequent series of studies focusing on the thermal response of steel I-girder IAB superstructures was carried out in Illinois using both long-term field monitoring (LaFave et al. 2017, 2021) and numerical simulations (LaFave et al. 2016a; 2016b). These investigations expanded the scope of the previous Illinois research from substructure behavior to superstructure response under thermal loading and provided a comprehensive evaluation of the influence of bridge geometry and structural details on thermal performance. The monitored bridges were used to validate numerical models that were subsequently employed in parametric studies investigating the effects of bridge skew, span length, number of spans, and foundation characteristics.

Additional insight into the thermal behavior of IABs has also been provided by other researchers. Pugasap et al. (2009) investigated the long-term behavior of integral abutment bridges using finite element models and field monitoring of three full-scale nonskewed prestressed concrete bridges in Pennsylvania. The models incorporated seasonal temperature changes, concrete creep and shrinkage, soil-structure interaction, and time-dependent effects, and were validated using bridge displacements and abutment soil pressure. Accumulation of abutment displacements was observed to be significant during the first three years; additionally, the ratio of long-term to short-term predicted abutment displacements ranged from 1.5 to 2.3, indicating that long-term bridge behavior should not be neglected in design. The field data from this study was later reported and summarized by Kim and Laman (2012), who documented maximum tensile axial forces of 11,685 kN (2,627 kips) in a 52.4 m (172 ft) three-span bridge, compressive axial forces of 6,935 kN (1,559 kips) in a 128 m (420 ft) four-span bridge, and bending moments of 8,270 kN-m (6,100 kip-ft) in a 34.7 m (114 ft) single-span bridge.

Hoffman and Phares (2014) investigated the influence of thermal loading on horizontally curved steel I-girder bridges with integral abutments. Although the behavior of curved bridges differs from that of straight skewed bridges, their findings provide useful insight into the effects of restraint on thermally induced stresses. The study employed a validated 3D finite element model of a three-span bridge with fixed pier supports and integral abutments to evaluate superstructure response under thermal loading. The uniform temperature change was determined following Procedure “A” of the AASHTO Specifications (2010). The analyses showed that the largest thermally induced stresses occurred in the bottom flange at the fixed pier locations, where lateral flange bending was the primary contributor due to the restraint against lateral girder movement provided by the fixed piers. Under a uniform temperature change, the unfactored thermally induced bottom flange lateral bending stress reached approximately 13.8 MPa (2.0 ksi).

Oesterle et al. (2007) investigated the thermal behavior of continuous composite steel jointless bridges subjected to both uniform annual temperature changes and nonlinear thermal gradients. The study evaluated the longitudinal restraint forces associated with uniform temperature changes and performed a parametric investigation of continuous composite steel I-girder bridges considering two, three, four, and multispan configurations, rolled W36x135, W36x182, and W36x230 sections, as well as welded plate girders, all with a nominal girder depth of 91.4 cm (36 in.), girder spacings ranging from 1.2 to 4.8 m (4 to 16 ft), and deck thicknesses between 17.8 and 27 cm (7 to 10.6 in.). The uniform temperature analyses showed that bridge contraction is generally the critical loading condition, with the maximum longitudinal restraint force limited by yielding of the abutment piles

and sliding of the approach slab, which establishes an upper bound on the thermal restraint force. The thermal gradient analyses combined stresses induced by nonlinear thermal gradients with restraint stresses resulting from the annual uniform temperature change, assuming that approximately two-thirds of the maximum longitudinal expansion could coincide with the maximum thermal gradient. The highest compressive stresses occurred in the bottom flange over the interior pier of two-span bridges, where thermal-gradient stresses ranged from 82.7 to 139.3 MPa (12 to 20.2 ksi), whereas longitudinal restraint contributed only 6.9 to 10.3 MPa (1 to 1.5 ksi).

These studies have advanced the understanding of the thermal behavior of integral abutment bridges and established the importance of considering soil-structure interaction, bridge geometry, and long-term thermal effects in design. Nevertheless, relatively limited attention has been devoted to localized girder demands produced by thermal restraint, particularly thermally induced weak-axis lateral bending stresses in skewed steel I-girder bridges. In addition, comprehensive validation of 3D finite element models against long-term field measurements remains limited. The present study addresses these gaps through extensive long-term field monitoring of representative Illinois skewed steel I-girder bridges, development and validation of detailed 3D finite element models, and a comprehensive investigation of thermally induced girder demands, including both axial and weak-axis lateral bending stresses. The validated numerical models are used to evaluate the influence of bridge skew, abutment type, cross-frame configuration, and girder location on the magnitude and distribution of thermally induced stresses, as well as to identify the critical bridge regions where the maximum demands occur.

Field monitoring results presented in Chapter 4 demonstrated that substantial lateral bending stresses can develop in both stub abutment and integral abutment bridges solely due to thermal demands. In the monitored stub abutment bridge, thermally induced weak-axis bending stress range at an exterior girder span location reached approximately 17 MPa (2.4 ksi) over the monitoring period, with the response exhibiting a pronounced asymmetry between increasing and decreasing temperatures. This value represents the total recorded stress range, encompassing stresses developed due to both uniform temperature changes and thermal gradients. Typically, only approximately half of the range would correspond to lateral bending stress developing in a single direction at a given time; however, the asymmetry in the response could indicate that a significantly larger portion of the recorded range may develop in a single lateral bending direction. Even larger thermally induced lateral bending stress ranges were observed in the monitored integral abutment bridge, where maximum weak-axis bending stress range of approximately 30 MPa (4.3 ksi) near the abutment and 18 MPa (2.5 ksi) near midspan were recorded on exterior girders. These observations indicated that thermal demands alone can contribute a substantial portion of the AASHTO-suggested lateral bending stress limits for skewed bridges, despite such limits being intended to account for the combined effects of all unfactored loading conditions. Furthermore, these lateral bending stresses are higher than those found by Hoffman and Phares (2014) in curved bridges. Additional exploration on bottom flange lateral bending stresses considering a more extensive set of geometric bridge parameters is carried out in this chapter.

Field observations further indicated that the dominant mechanisms governing thermal response may differ depending on the stress component considered. While weak-axis bending behavior frequently

exhibited significant short-term deviations associated with thermal gradients, axial stress response appeared to be governed primarily by long-term bridge expansion and contraction behavior. Maximum recorded axial stress range reached approximately 32 MPa [4.6 ksi] in the monitored stub abutment bridge and 39 MPa (5.6 ksi) in the monitored integral abutment bridge (16 MPa [2.3 ksi] and 19 MPa [2.8 ksi] in a single direction, respectively), which exceed the 6.9 to 10.3 MPa (1 to 1.5 ksi) longitudinal restraint stresses reported by Oesterle et al. (2007). Nevertheless, the measured axial response exhibited comparatively limited short-term deviations from the dominant long-term temperature-stress relationship.

Oesterle et al. (2007) reported bottom-flange strong-axis bending stresses due to nonlinear thermal gradients ranging from 82.7 to 139.3 MPa (12 to 20.2 ksi). For comparison with the field measurements, one-half of the maximum error range in Table 11 and Table 12 may be considered representative of the maximum thermal gradient stress developing in a single direction. On this basis, the maximum field-observed strong-axis bending stresses associated with short-term thermal gradients were approximately 21 MPa (3.1 ksi) in both monitored bridges, much lower than those reported by Oesterle et al. (2007). Note, however, that the field measurements are limited to the instrumented cross-sections, and larger thermal gradient-induced stresses may occur at other locations within the bridge.

These findings collectively motivate the need for a broader investigation into thermally induced stress demands in skewed steel I-girder bridges and provide important context for the numerical parametric studies presented in this chapter, which evaluate the complete spatial distribution of stresses throughout the bridge system rather than only at discrete instrumented locations. Because the numerical models primarily capture the long-term bridge response associated with uniform thermal expansion and contraction, the mean absolute error (MAE) and maximum error analyses presented in Chapter 4 provide approximate estimates of the short-term thermal-gradient-induced stresses that are not explicitly represented in the numerical simulations, particularly for weak-axis bending response.

PARAMETRIC STUDY RESULTS

Stress results presented in this chapter were extracted and processed considering all interior and exterior girders within each analyzed bridge configuration. For plots where stresses are separated by bridge location (e.g., abutment, span, pier, acute corner, obtuse corner), an independent maximum stress value was determined for each region within each bridge configuration. Definitions of the span, pier, and abutment regions are provided in Chapter 3.

Total Stress

Figure 71 presents the envelope and interquartile range of maximum total girder stresses for the analyzed bridge configurations, separated by abutment type. Integral abutment bridges consistently develop substantially larger total stresses than stub abutment bridges across the full range of analyzed skew angles, reflecting the greater restraint forces associated with integral support conditions.

For stub abutment bridges, total stress magnitudes remain relatively stable with skew angle. Envelope values increase from approximately 25 MPa (3.6 ksi) at low skew angles to approximately 55 MPa (8 ksi) at 70° skew, while IQR values remain comparatively low, generally below approximately 18 MPa (2.6 ksi), with demands being mostly symmetric in compression (negative values) and tension (positive values). Integral abutment bridge stresses are observed to be considerably higher, with envelope values approaching 85 MPa (12 ksi) at low skew angles and gradually decreasing as skew increases beyond approximately 45°.

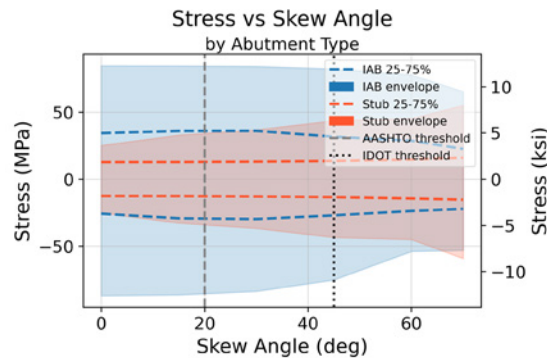


Figure 71. Graph. Envelope and IQR of maximum total stress vs. skew (by abutment type).

Axial Stress

Figure 72 presents the envelope and IQR of girder axial stresses for both interior and exterior girders, separated by abutment type. The response trends for interior and exterior girders are very similar, indicating that axial stress development is not strongly influenced by girder location. Also, the response is observed to be mostly symmetric both in tension and compression. For integral abutment bridges, the axial stress envelope generally ranges from approximately 21 MPa (3 ksi) to 24 MPa (3.5 ksi), while the IQR remains between approximately 14 MPa (2 ksi) and 18 MPa (2.6 ksi) for both interior and exterior girders. Stub abutment bridges develop lower axial stresses, with the envelope reaching approximately 16 MPa (2.3 ksi), and the IQR remaining between 10 MPa to 12 MPa (1.5 ksi to 1.8 ksi). Axial stress response does not exhibit a pronounced increase with skew angle, with both girder types maintaining relatively stable stress magnitudes throughout the analyzed skew range.

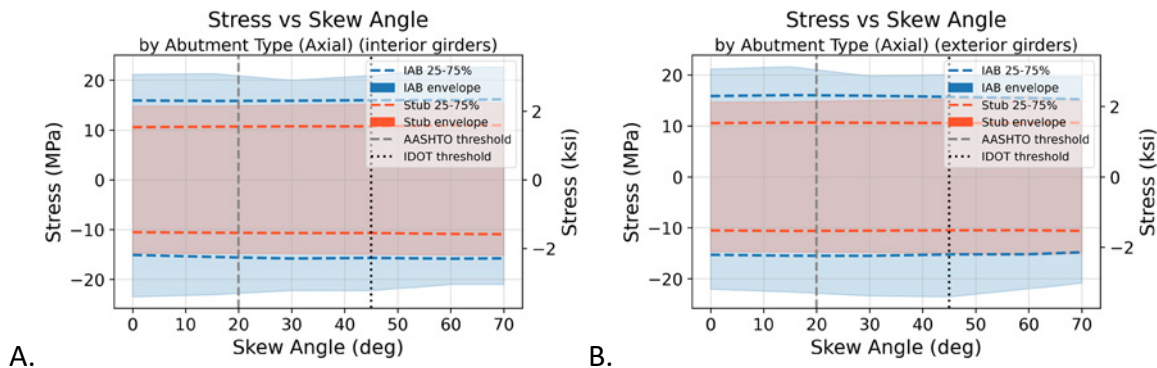


Figure 72. Graph. Envelope and IQR of maximum axial stress vs. skew (by abutment type) for (A) interior girders and (B) exterior girders.

Figure 73 presents the relationship between bridge length and maximum axial stress for both interior and exterior girders. Similar to the trends observed with skew angle, bridge length does not appear to have a strong influence on the maximum thermally induced axial stresses for either abutment type.

This relatively weak dependence on bridge length might be associated with the restraint characteristics represented in the finite element models. In particular, the foundation response of the integral abutment piles was modeled using nonlinear p - y curves, which provide a progressively softening lateral resistance as pile displacement increases while assuming elastic behavior of the steel piles. This observation should be interpreted with caution since the analytical models were validated using thermal responses corresponding to the range of temperatures observed during field monitoring, whereas the parametric analyses employ the AASHTO (2024) Procedure “A” extreme uniform temperature changes.

Additionally, the analyzed bridge matrix includes configurations exceeding the bridge lengths currently permitted under typical IDOT integral abutment bridge provisions (186 m [610 ft]). The largest bridge considered has a total length of approximately 200 m (650 ft), with maximum span length approaching 60 m (200 ft). Although the present analytical results suggest that axial stresses do not continue to increase substantially with bridge length within the investigated range, these findings should be interpreted within the assumptions of the numerical modeling approach and the adopted foundation representation.

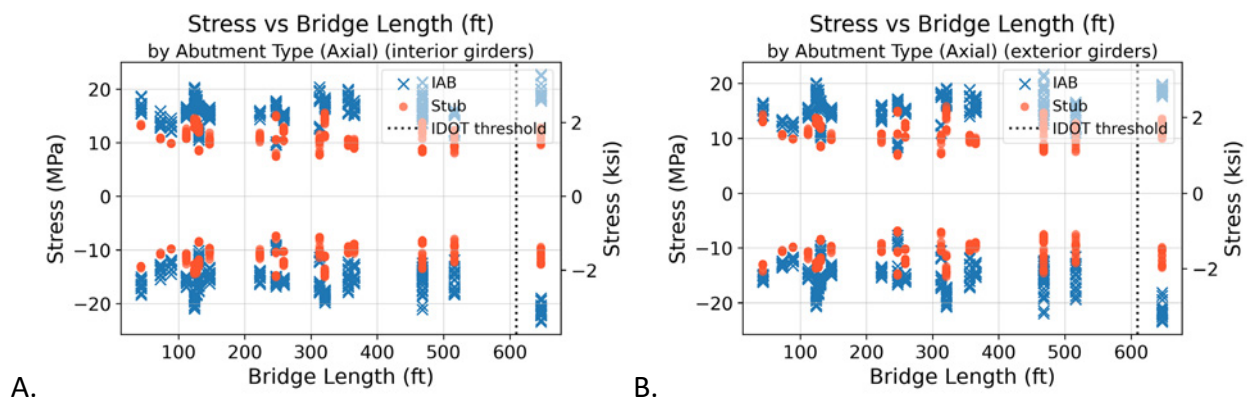


Figure 73. Graph. Envelope and IQR of maximum axial stress vs. total bridge length (by abutment type) for (A) interior girders and (B) exterior girders.

Bottom-Flange Weak-Axis Bending Stress

Figure 74 present the envelope and IQR of bottom-flange weak-axis bending stresses for both interior and exterior girders, separated by abutment type. In contrast with axial stresses, weak-axis bending response exhibits a pronounced dependence on skew angle for both girder types. For both integral and stub abutment bridges, envelope and IQR values generally increase with skew angle, indicating that skew effects influence the development of lateral bending stresses under thermal demands.

Interior and exterior girders exhibit very similar overall trends, with both showing increasing weak-axis bending stresses as skew increases and with IABs generally developing larger stress demands

than stub abutment bridges. For both girder types, envelope stresses increase from approximately 13–21 MPa (2–3 ksi) at low skew angles to values approaching 38 MPa (5.5 ksi) at high skew angles. IQR values also increase with skew and remain larger for IAB configurations, reaching approximately 13 MPa (1.9 ksi) for interior girders and approximately 11 MPa (1.6 ksi) for exterior girders at high skew angles, while stub abutment bridge IQR values remain below approximately 8 MPa (1.2 ksi). These results indicate that integral support restraint and increase in skew further amplify the development of lateral bending stresses.

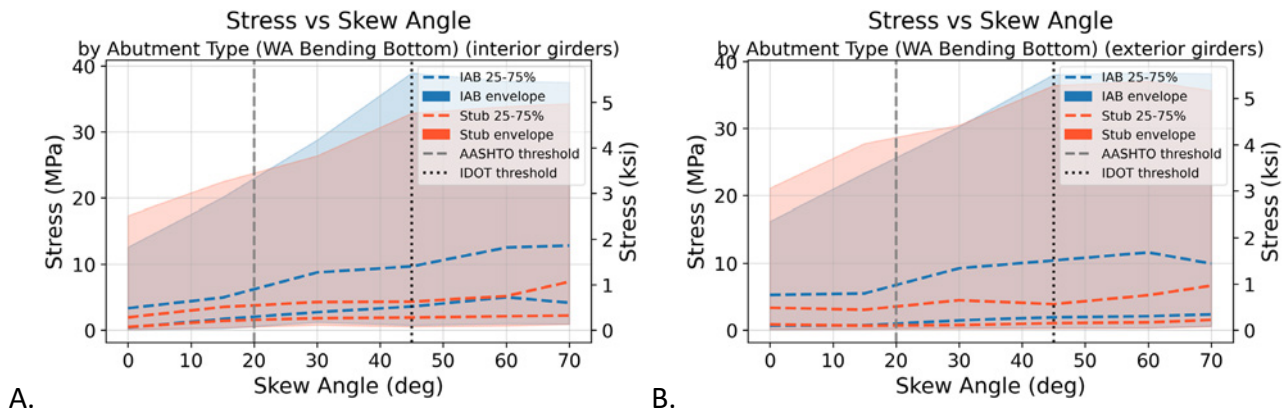


Figure 74. Graph. Envelope and IQR of maximum lateral bending stress vs. skew (by abutment type) for (A) interior girders and (B) exterior girders.

Figure 75 presents the envelope and IQR of bottom-flange weak-axis bending stresses for both interior and exterior girders, separated by bridge location, to better identify where lateral bending stresses develop along the bridge length. Similar trends are observed for both girder types. Weak-axis bending stresses near the abutments and piers increase with skew angle, while stresses within span regions remain comparatively small throughout the analyzed skew range. This behavior suggests that thermal weak-axis bending stresses are strongly influenced by restraint conditions and transverse load transfer mechanisms concentrated near support regions.

Pier-region stresses remain the largest across the full range of analyzed skew angles, with envelope values increasing from approximately 18 MPa (2.6 ksi) at low skew to values approaching 38 MPa (5.5 ksi) at high skew angles. The largest pier-region lateral bending demands are primarily associated with bridges with fixed bearings and rigid pier supports, where the greater restraint against thermal movement promotes the development of larger weak-axis bending stresses. Abutment-region stresses also increase with skew, reaching envelope values of approximately 30 MPa (4.5 ksi) at moderate-to-high skew angles. In contrast, midspan stresses remain comparatively small throughout the analyzed skew range, with envelope values generally below approximately 10 MPa (1.5 ksi). IQR values follow similar trends, with both abutment and pier regions exhibiting substantially larger variability than span regions.

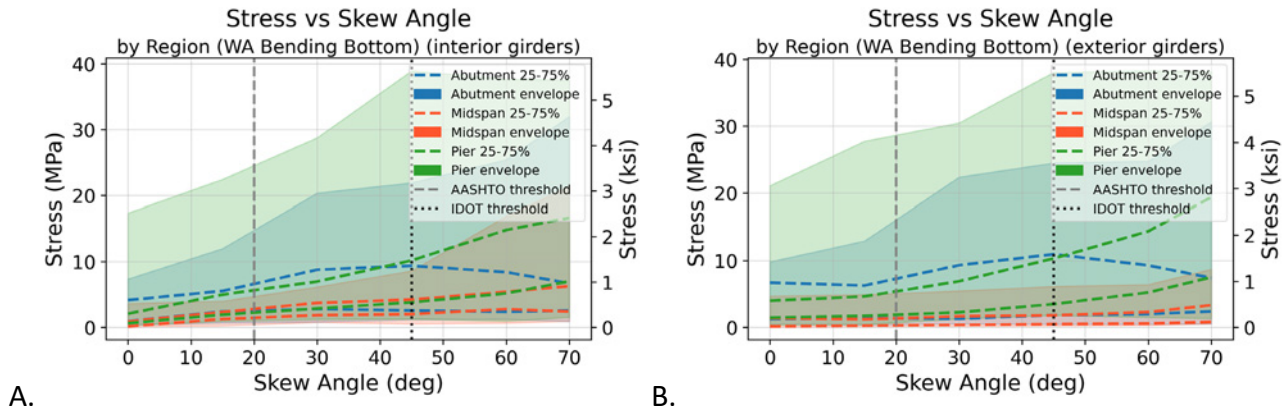


Figure 75. Graph. Envelope and IQR of maximum lateral bending stress vs. skew (by bridge location) for (A) interior girders and (B) exterior girders.

Figure 76 further examines the abutment-region stresses identified in Figure 75 by separating the response into acute and obtuse corners. Only data corresponding to the abutment regions shown in Figure 75 were included in this comparison. For each bridge, the maximum stresses at the acute and obtuse corners were identified independently, allowing both corner types to contribute to the summarized response. The results show that the largest thermally induced bottom-flange weak-axis bending demands consistently develop at the obtuse corners, with the difference between the two corner types becoming more pronounced as skew angle increases.

The interquartile ranges exhibit a similar trend, with greater variability observed at the obtuse corners than at the acute corners. The increasing disparity between the two corner types with increasing skew suggests that the restraint-induced load path becomes progressively more asymmetric as skew increases, resulting in a greater concentration of lateral bending stresses at the obtuse corner. These observations further support the results presented in Figure 75, where the abutment regions were shown to be among the critical locations for the development of thermally induced weak-axis bending stresses.

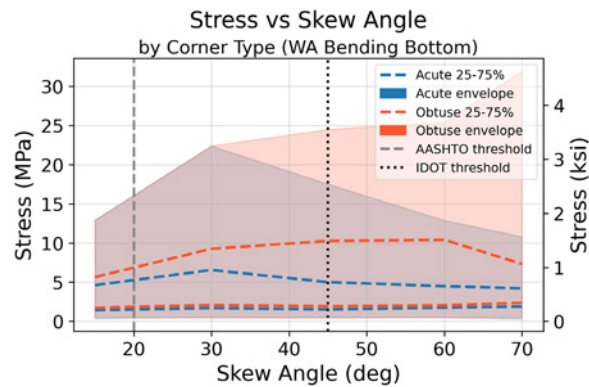


Figure 76. Graph. Envelope and IQR of maximum lateral bending stress vs. skew (by cross-frame type)

Figure 77 illustrates the influence of cross-frame configuration on thermally induced bottom-flange weak-axis bending stresses. Among the configurations considered, bridges with X-staggered cross-frames consistently develop the largest lateral bending stresses, with both the envelope and IQR increasing as skew angle increases. In contrast, channel-staggered and channel-parallel configurations produce lower stress demands and exhibit a more limited sensitivity to skew angle. These results indicate that cross-frame configuration can significantly influence the development of thermally induced lateral bending stresses and should be considered when evaluating the response of skewed steel I-girder bridges.

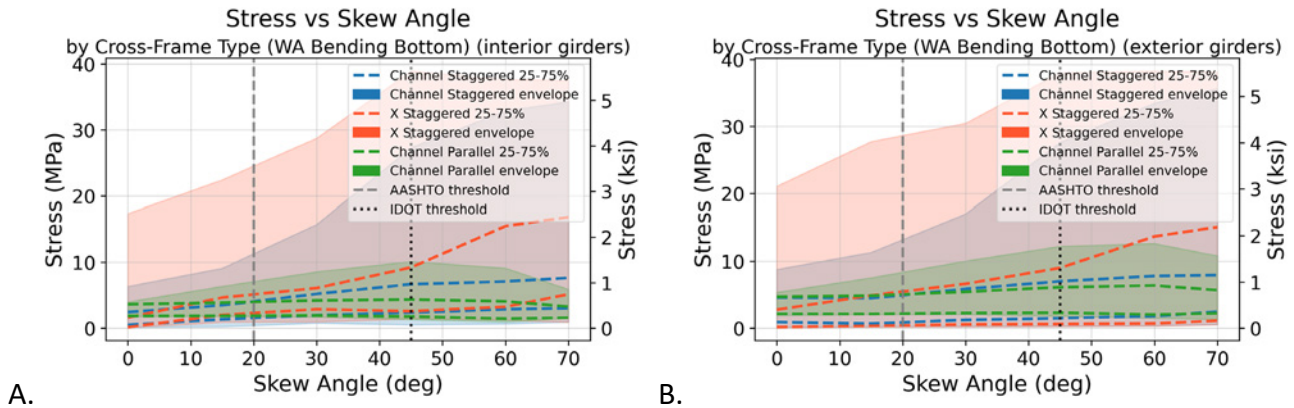


Figure 77. Graph. Envelope and IQR of maximum lateral bending stress vs. skew (by cross-frame type) for (A) interior girders and (B) exterior girders.

Overall, the results presented in this chapter demonstrate that thermally induced lateral bending stresses in skewed steel I-girder bridges can become substantial, particularly in IAB configurations and at high skew angles. While axial stresses remained relatively stable across the analyzed skew range and showed limited sensitivity to girder location, weak-axis bending stresses exhibited a pronounced dependence on skew angle, support restraint, and bridge location. The largest thermal lateral bending stresses were concentrated near pier and abutment regions, where restraint effects are most significant, while stresses within span regions remained comparatively small. Comparisons with current AASHTO recommendations indicate that the computed thermal-induced lateral bending stresses can represent a substantial portion of the stress levels currently recognized for skew-related lateral bending behavior, particularly for IAB configurations. Furthermore, the results demonstrate that meaningful lateral bending stresses develop even below the AASHTO applicability threshold of 20° skew and below the 45° skew threshold currently adopted by IDOT for explicit consideration of lateral bending stresses, suggesting that thermal lateral bending effects may warrant consideration even within bridge geometries currently permitted by IDOT practice without such evaluation.

CHAPTER 6: SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

Volume II of this report series investigated the structural demand and load distribution behavior of skewed steel I-girder bridges with both stub abutments and integral abutments under construction loading and in-service live load. The study combined field monitoring of two newly constructed bridges in Champaign, Illinois—Mattis-74 (41° skew, stub abutments) and Mattis-57 (45° skew, integral abutments)—with detailed 3D finite element analyses and numerical parametric studies. Field measurements and numerical simulations were post-processed into strong-axis bending, lateral bending, and axial stress components, allowing the mechanisms governing skew-related bridge behavior to be identified and evaluated. The modeling approaches were validated through good agreement between measured and simulated responses under both deck placement and isolated truck loading.

The results demonstrated that skew significantly influences flange lateral bending behavior through both global bridge rotation about skewed supports and local transverse load transfer through cross-frames. During deck placement, lateral bending stresses near bridge supports were strongly governed by skew-induced global rotation, while span-region stresses were influenced primarily by transverse load transfer through staggered cross-frames. Under truck live load, critical strong-axis bending stresses occurred at girders closest to the applied load path, whereas critical lateral bending stresses consistently developed at cross-frame locations. Interior girders were generally more critical than exterior girders regarding bottom-flange lateral bending stresses along bridge spans, supporting the reduced exterior girder lateral bending stresses permitted by AASHTO for skewed bridges with staggered cross-frame layouts.

Parametric studies further showed that skew angle, bridge width, support condition, and first cross-frame location strongly influence lateral bending response. Strong-axis bending stresses generally decreased slightly with increasing skew angle, while lateral bending stresses increased substantially with skew. Narrow bridges exhibited smaller lateral bending response than wider bridges, and the distance between the bridge support and first intermediate cross-frame was identified as a critical parameter controlling stress concentrations near supports. Regarding flange lateral bending stresses, the findings supported AASHTO recommendations distinguishing between interior and exterior girders, while also emphasizing the importance of refined analysis methods for bridges with large skew angles or significant transverse load transfer effects. Finally, field measurements indicated that the AASHTO dynamic load allowance (DLA) requirement of 33% was generally conservative, although localized exceedances were observed near obtuse corners in the integral abutment bridge.

NUMERICAL SIMULATION

A detailed 3D finite element (FE) modeling framework was developed to simulate the behavior of skewed steel I-girder bridges subjected to live and thermal loads. The numerical approach explicitly modeled girders, concrete deck, cross-frames, stiffeners, bearings, abutments, piers, and soil-structure interaction effects using shell, frame, connector, and spring elements within ABAQUS.

Particular emphasis was placed on accurately representing boundary conditions and restraint mechanisms governing bridge response, especially for integral abutment bridges (IABs), where thermal demands are strongly influenced by interaction between the superstructure, substructure, bearings, and surrounding soil. The model validation process demonstrated that simplified support assumptions can significantly alter predicted stress demands, particularly for thermal loading conditions, emphasizing the importance of realistic representation of support flexibility in bridge FE models.

Key observations and practical implications of the modeling approach are summarized below:

- Explicit representation of bearings, piers, and substructure flexibility was critical for realistic prediction of thermal response and stress redistribution.
- Modeling fixed bearings with realistic stiffness behavior produced more accurate bridge response than idealized pinned restraints, which generated unrealistically high lateral bending stresses.
- Explicit modeling of piers prevented artificial over-restraint of the bridge system during thermal expansion and contraction.
- Sensitivity studies demonstrated that pile foundation stiffness is one of the most influential parameters governing thermally induced response in IABs. Accounting for temperature cyclic loading in lateral pile stiffness significantly improved agreement between FE-predicted and field-measured stresses.
- Modeling of approach slabs improved prediction of live load response near bridge ends and provided more realistic simulation of support-region boundary conditions.
- Positioning the shell elements modeling an abutment at the abutment face significantly improved prediction of local stresses near supports by more accurately representing the distance between the abutment and first cross-frame.

Validation against monitored bridge response demonstrated that the FE models adequately reproduced the dominant long-term thermally induced behavior of the monitored bridges. Excellent agreement was obtained between FE-predicted and measured axial stresses, crackmeter displacements, and overall temperature-stress relationships, providing confidence that the implemented modeling approach successfully captures the primary mechanisms governing restrained thermal behavior. Weak-axis (lateral) bending stresses exhibited larger short-term deviations between FE and field measurements, suggesting that thermal gradients—which were not explicitly simulated—have a greater influence on lateral bending response than on axial behavior. Nevertheless, the FE models consistently reproduced the dominant long-term trends and overall magnitude of the measured response, confirming the suitability of the modeling framework for evaluating thermally induced behavior of skewed steel I-girder bridges.

LIVE LOAD PARAMETRIC STUDY

The live load parametric study demonstrated that skew angle, support condition, and cross-frame configuration strongly influence development and spatial distribution of girder flange lateral bending stresses in steel I-girder bridges. While total stress response showed substantial scatter across all skew angles, the dominant influence of skew was observed in the lateral bending component rather than in the overall total stress magnitude. Lateral bending stresses generally increased with skew angle for both interior and exterior girders, with interior girders consistently developing larger demands than exterior girders. Envelope interior girder lateral bending stresses approached approximately 40–50 MPa (6–7 ksi) at high skew angles, while exterior girder stresses generally remained below approximately 21 MPa (3 ksi).

Key observations and practical implications from the live load parametric studies are summarized below:

- Significant flange lateral bending stresses (up to 4.1 ksi on interior girders and 3 ksi on exterior girders) developed even below the AASHTO applicability threshold of 20° skew and below the 45° threshold currently used by IDOT for explicit consideration of lateral bending stresses.
- At low skew angles, the largest lateral bending stresses were concentrated near abutment regions, indicating that support restraint and transverse load paths near supports govern response even for moderately skewed bridges.
- The location of governing lateral bending stress varied with bridge geometry and support conditions. For integral abutment bridges, lateral bending near abutments typically governed response, while pier-region stresses became increasingly important at higher skews and in stub abutment bridges.
- Midspan lateral bending stresses rarely governed response for exterior girders and remained substantially smaller than stresses near supports and pier regions.
- Cross-frame type strongly influenced lateral bending response. X-type staggered systems consistently produced larger stresses than channel-staggered configurations, with interior girder IQR values approximately 60–80% larger across all skew angles. Channel-staggered systems showed consistently lower stress levels, suggesting current design requirements may be conservative for these configurations.
- Compliance with the AASHTO $4b_f$ first cross-frame spacing recommendation generally resulted in lower lateral bending stress demands near supports.
- Interpretation of AASHTO exterior girder lateral bending recommendations was shown to significantly affect assessment of code compliance. The interpretation of 14 MPa (2 ksi) within spans and 52 MPa (7.5 ksi) near supports resulted in computed envelope stresses remaining generally within prescribed limits. However, applying a uniform 14 MPa (2 ksi) limit along the entire bridge length would result in exceedances near both supports and pier regions, emphasizing the importance of location-specific treatment of exterior girder lateral bending stresses.

Any modification to existing lateral bending stress recommendations should be considered within the broader context of combined demand effects. In particular, lateral bending stresses induced by live load, as examined in Chapter 3, would need to be evaluated with lateral bending contributions from dead load. Furthermore, for IABs, thermal loading plays a critical role due to restraint of longitudinal bridge movements at the abutments, which induces substantial axial and lateral bending demands throughout the superstructure, as demonstrated in Chapter 5. A comprehensive assessment accounting for the combined effects of live load, dead load, and thermal actions—particularly for integral abutment configurations—is necessary before refinement of current code-recommended lateral bending limits.

BRIDGE RESPONSE UNDER THERMAL LOADING

Field monitoring of the two instrumented bridges demonstrated that long-term thermal loading produces substantial stresses, displacements, and rotations in skewed steel I-girder bridges. The monitored bridges, located in close proximity and subjected to essentially identical environmental conditions, provided a unique opportunity to isolate the influence of abutment type on thermal behavior. Over the monitoring period presented in this report volume, from September 2021 through December 2025, the bridges experienced a total recorded mean temperature variation of approximately 60°C (from –23°C to 37°C), allowing evaluation of both long-term seasonal response and short-term thermal gradient effects.

Key observations and practical implications from the field-monitoring study are summarized below:

- Significant thermal gradients were observed during warmer periods, both vertically through the bridge depth and horizontally across the bridge width. Exterior bottom flange gauges experienced localized heating effects associated with direct sunlight exposure, producing distinctive short-term thermal behavior not present during colder periods.
- Previously reported strain gauge “deviations” observed in earlier Illinois monitoring studies were confirmed to be associated with strain gauge malfunction rather than structural behavior. Side-by-side installation of new gauges adjacent to anomalous gauges demonstrated that the observed long-term drift behavior was not mechanically consistent with the surrounding instrumentation response.
- Raw stress data indicated substantially larger thermal stress ranges in the IAB compared to the stub abutment bridge. Exterior girder stress ranges in the IAB reached up to 107 MPa (15.5 ksi) near the abutments and 80 MPa (11.4 ksi) near span regions, while the largest stress ranges in the stub abutment bridge remained below approximately 48 MPa (7 ksi). These reported values correspond to total stress ranges over the monitoring period, with approximately one-half of the range representing the magnitude of the maximum tensile or compressive stress relative to the mean response. These measurements correspond only to discrete instrumented locations throughout the bridges and should not be interpreted as definitive global maximum bridge stresses.

- Strong-axis bending stresses were generally larger within spans and near abutments than near piers, with the monitored IAB developing larger stress ranges than the stub abutment bridge. Maximum stress ranges reached approximately 57 MPa (8.3 ksi) in the IAB and 42.5 MPa (6.2 ksi) in the stub abutment bridge. Short-term scatter around the dominant temperature-stress trend indicated significant influence of vertical thermal gradients on flexural response.
- Thermally induced weak-axis (lateral) bending stresses reached substantial magnitudes, particularly in the IAB. Maximum lateral bending stress ranges reached approximately 30 MPa (4.3 ksi) near abutments and 18 MPa (2.5 ksi) near midspan on exterior girders of the monitored IAB, while the stub abutment bridge reached approximately 17 MPa (2.4 ksi). These results indicate that thermal variations alone can contribute a substantial portion of the AASHTO-recommended lateral bending stress limits for skewed bridges.
- The weak-axis bending response exhibited noticeable asymmetry during heating and cooling cycles, suggesting thermal gradient effects can produce strong lateral bending behavior.
- Axial stresses were strongly correlated with long-term temperature variation and exhibited comparatively small short-term deviations from the dominant temperature-stress trend. Maximum axial stress ranges reached approximately 39 MPa (5.6 ksi) in the monitored IAB and 32 MPa (4.6 ksi) in the stub abutment bridge.
- Regression analyses show that thermal gradients might induce substantial short-term stress demands, particularly for strong-axis and weak-axis bending stresses, while axial response exhibited comparatively smaller dispersion. The monitored IAB generally developed larger scatter magnitudes than the stub abutment bridge.
- Tiltmeter data showed a clear temperature-rotation relationship in both bridges. However, the relationship was substantially more complex in the IAB, suggesting strong interaction between the superstructure, abutments, pile foundations, and surrounding soil.

Overall, the field-monitoring results demonstrated that thermal loading induces complex coupled axial, flexural, lateral bending, displacement, and rotational behavior in skewed steel I-girder bridges. The monitored IAB consistently developed larger stresses and more complex thermal response than the stub abutment bridge, highlighting the importance of support restraint and soil-structure interaction effects in governing long-term bridge behavior under thermal loading.

THERMAL LOAD PARAMETRIC STUDY

Thermally induced stresses in skewed steel I-girder bridges were investigated through a comprehensive parametric study using validated 3D FE models. Particular emphasis was placed on axial stresses and bottom-flange weak-axis bending stresses resulting from restrained thermal expansion and contraction in both stub abutment and IAB configurations. The analyses considered bridge geometries representative of current Illinois practice, including bridges exceeding typical IDOT IAB length limits, and evaluated the influence of skew angle, support restraint, and bridge location on thermally induced response.

Key observations and practical implications from the thermal parametric studies are summarized below:

- Integral abutment bridges consistently developed substantially larger total stresses than stub abutment bridges across the full range of analyzed skew angles due to restrained thermal expansion and contraction at the supports. Total stress envelope values approached approximately 85 MPa (12 ksi) in IABs, while stub abutment bridge envelope stresses remained below approximately 55 MPa (8 ksi). Interior and exterior girders exhibited very similar behavior for all response components, unlike behavior under live loads, where behavior was clearly different for exterior and interior girders.
- Axial stress response exhibited limited sensitivity to skew angle and girder location. Interior and exterior girders developed very similar axial stress magnitudes, indicating that thermal axial response is governed primarily by global restraint effects rather than localized transverse load transfer mechanisms. Despite the large bridge geometries analyzed—including bridges approaching approximately 200 m (650 ft) total length and 60 m (200 ft) span length—maximum axial stress envelopes in IABs generally remained between approximately 21–24 MPa (3–3.5 ksi). This apparent weak dependence on bridge length may be associated with the nonlinear foundation restraint represented by the p - y curves adopted in the numerical models. However, this interpretation should be considered with caution, as the models were validated against field-monitored temperature variations rather than the extreme uniform temperature changes considered in the parametric study.
- Bottom-flange weak-axis bending stresses exhibited a pronounced dependence on skew angle and support restraint conditions. Envelope stresses increased from approximately 13–21 MPa (2–3 ksi) at low skew angles to approximately 38 MPa (5.5 ksi) at high skew angles for both interior and exterior girders (IABs and stub abutment bridges).
- Integral abutment bridges generally developed larger weak-axis bending stresses and larger IQR values than stub abutment bridges, indicating that support restraint significantly amplifies thermally induced lateral bending behavior.
- Thermally induced lateral bending stresses were concentrated primarily near pier and abutment regions, where restraint effects are most significant. The largest pier-region stresses were generally associated with bridges incorporating fixed bearings and rigid pier supports. Pier regions developed the largest lateral bending stresses across the analyzed skew range, reaching envelope values approaching 38 MPa (5.5 ksi), while abutment-region stresses approached approximately 30 MPa (4.5 ksi). In contrast, span-region stresses generally remained below approximately 10 MPa (1.5 ksi).
- Within the abutment regions, thermally induced weak-axis bending stresses consistently concentrated at the obtuse corners of skewed bridges. The disparity between obtuse and acute corner demands increased with skew angle, indicating that restraint-induced lateral bending becomes progressively more asymmetric as bridge skew increases.

- Cross-frame configuration influenced thermally induced lateral bending stresses. Bridges with X-staggered cross-frames consistently developed the largest weak-axis bending stresses, whereas channel-staggered and channel-parallel configurations produced lower demands and exhibited reduced sensitivity to skew angle.
- Meaningful thermally induced lateral bending stresses develop even below the 20° skew threshold currently used by AASHTO for explicit consideration of lateral bending behavior and below the 45° skew threshold currently adopted by IDOT.
- Comparisons with current AASHTO recommendations indicated that thermally induced lateral bending stresses alone can represent a substantial portion of the stress levels currently recognized for skew-related lateral bending behavior, particularly in IAB configurations and near support regions.

SUMMARY OF GOVERNING GIRDER STRESS DEMANDS

The preceding sections summarized the main findings from the field monitoring program, live load parametric study, and thermal parametric study. To provide an overall perspective of these results, the governing stress demands identified throughout this study are consolidated in Tables 20 and 21 for interior and exterior girders, respectively. These tables summarize the absolute maximum stress envelopes obtained from the field measurements and finite element parametric studies, representing the largest demands observed among the bridges and loading conditions considered. Therefore, the reported values should be interpreted as upper-bound responses, whereas the stresses experienced by a typical bridge are generally expected to be lower, as discussed throughout this document. By presenting the governing axial, strong axis bending, weak axis bending, and total stresses in a single format, the tables facilitate direct comparison of the relative contribution of the different stress components and loading mechanisms.

The weak axis bending results provide useful context for evaluating current lateral bending stress recommendations. The AASHTO *LRFD Bridge Design Specifications* (2024) recommend considering lateral bending stresses for bridges with skews greater than 20°, whereas current IDOT practice considers lateral bending stresses only for bridges with skews greater than 45°. Under the AASHTO provisions, the recommended lateral bending stress is 69 MPa (10 ksi) for interior girders regardless of location, while the suggested lateral bending stress for exterior girders is 52 MPa (7.5 ksi) near support regions and 14 MPa (2 ksi) within span regions. Similarly, current IDOT practice adopts a uniform 69 MPa (10 ksi) lateral bending stress for both interior and exterior girders, independent of location. It should be recognized, however, that these recommended values were established to account for dead load and live load effects and do not explicitly consider thermally induced lateral bending stresses. Consequently, the lateral bending values summarized in Tables 20 and 21 should not be interpreted as direct comparisons with the code-recommended values, since they exclude dead load contributions while explicitly including thermal effects.

The live load analyses generally produced the largest interior girder weak-axis bending demands, whereas the thermal analyses generated comparable demands for both interior and exterior girders, demonstrating that thermal restraint represents a significant source of lateral bending in integral

abutment bridges. The influence of cross-frame configuration is also evident, with X-staggered cross-frames consistently producing larger weak-axis bending demands than channel-staggered cross-frames, although this is more evident when looking at the IQR values presented in Chapters 3 and 5. While the governing thermal demands generally occurred near support regions, appreciable weak-axis bending stresses were also observed within span regions.

Expressed relative to the current design recommendations, the maximum thermal weak axis bending corresponds to approximately 57% of the AASHTO and IDOT recommended lateral bending stress for interior girders and approximately 74% of the AASHTO recommended value for exterior girders near support regions. In contrast, the governing thermal weak axis bending demands within exterior girder span regions correspond to approximately 63% and 47% of the AASHTO recommended value for stub abutment and integral abutment bridges, respectively. The presented comparisons indicate that thermal loading can represent a substantial component of the overall lateral bending demand, particularly for integral abutment bridges. Additionally, these observations should be considered together with the corresponding dead load and live load contributions when evaluating overall girder lateral bending demands. The summarized results may provide additional insight for future evaluation of existing lateral bending stress recommendations and current IDOT design provisions, recognizing that restraint-induced thermal effects can become a significant contributor to the overall weak-axis bending response.

Table 20. Comparison of Governing Extreme Interior Girder Stress Demands

Response	Region	Field* Stub [MPa (ksi)]	Field* IAB [MPa (ksi)]	LL FE Stub [MPa (ksi)]	LL FE IAB [MPa (ksi)]	Thermal FE Stub [MPa (ksi)]	Thermal FE IAB [MPa (ksi)]
Max. Total Stress	Overall	20.25 (2.95)	36.5 (5.30)	92 (13.3)	67.2 (9.75)	59.2 (8.59)	86 (12.47)
Max. Axial Stress	Overall	16 (2.3)	18 (2.6)	–	–	15.6 (2.26)	23.5 (3.41)
Max. SA Bending	Overall	14.75 (2.15)	21 (3)	–	–	–	–
Max. WA Bending	Abutment	–	4 (0.6)	39.2 (5.68)	51.2 (7.42)	17.4 (2.52)	32 (4.64)
Max. WA Bending	Pier	2.75 (0.4)	1.5 (0.2)	32.1 (4.65)	32 (4.64)	34.3 (4.97)	38.9 (5.65)
Max. WA Bending	Span	3.5 (0.5)	3.25 (0.5)	33.7 (4.89)	33.6 (4.88)	11.2 (1.63)	21.4 (3.1)
Max. WA Envelope (X Stag.)	Overall	3.5 (0.5)	4 (0.6)	39.2 (5.68)	51.2 (7.42)	33.8 (4.91)	38.9 (5.65)
Max. WA Envelope (Channel Stag.)	Overall	–	–	27.6 (4.01)	31.3 (4.55)	34.3 (4.97)	32.9 (4.77)

Note: * indicates field data for which half of the measured stress range is reported. As noted in Chapter 4, the maximum single-direction bending stresses may be greater than half of the total range due to response asymmetry.

Table 21. Comparison of Governing Extreme Exterior Girder Stress Demands

Response	Region	Field* Stub [MPa (ksi)]	Field* IAB [MPa (ksi)]	LL FE Stub [MPa (ksi)]	LL FE IAB [MPa (ksi)]	Thermal FE Stub [MPa (ksi)]	Thermal FE IAB [MPa (ksi)]
Max. Total Stress	Overall	23.75 (3.45)	53.5 (7.75)	110 (16)	91 (13)	49.2 (7.14)	87 (12.61)
Max. Axial Stress	Overall	13.5 (1.95)	19.25 (2.8)	–	–	15.8 (2.3)	23.5 (3.41)
Max. SA Bending	Overall	21.25 (3.1)	28.5 (4.15)	–	–	–	–
Max. WA Bending	Abutment	–	15 (2.15)	18.7 (2.71)	20.6 (2.99)	16.4 (2.37)	30.6 (4.44)
Max. WA Bending	Pier	–	–	23.2 (3.37)	23 (3.33)	37.1 (5.37)	38.3 (5.55)
Max. WA Bending	Span	8.5 (1.2)	9 (1.25)	5 (0.72)	3.9 (0.57)	8.7 (1.26)	6.4 (0.93)
Max. WA Envelope (X Stag.)	Overall	8.5 (1.2)	15 (2.15)	23.2 (3.37)	23 (3.33)	37.1 (5.37)	38.3 (5.55)
Max. WA Envelope (Channel Stag.)	Overall	–	–	19.1 (2.77)	19.2 (2.79)	35.6 (5.17)	33.5 (4.86)

Note: * indicates field data for which half of the measured stress range is reported. As noted in Chapter 4, the maximum single-direction bending stresses may be greater than half of the total range due to response asymmetry.

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APPENDIX A: FIELD MONITORING DATA

The field-collected data from both monitored bridges (Mattis-74 [stub abutment bridge] and Mattis-57 [IAB]) is presented in this appendix. The data is presented from September 2021 to December 2025. The entirety of the data presented herein has been collected at a frequency of 0.5 Hz (2 secs). As discussed in Volume I (Zhou et al. 2026), the instrumentation used for data collection consists of strain gauges (mounted on steel I-girders, cross-frames, and embedded in concrete), tiltmeters (mounted at bridge ends), and crackmeters (mounted at bridge corners for the monitored stub abutment bridge, and between the approach and transition slab for the monitored IAB). Specific sensor type, model, resolution, and accuracy are presented in Volume I (Zhou et al. 2026).

The data is presented in: 1) raw stress, rotation, and displacement readings (for strain gauges, tiltmeters, and crackmeters, respectively), and 2) decomposed stress readings for girder cross-sections where enough strain gauges are installed to allow for stress decomposition; following the stress decomposition scheme outlined in Volume I (Zhou et al. 2026). For strain gauges mounted on steel, readings are multiplied by the modulus of elasticity of the steel I-girders ($E_s = 200 \text{ GPa}$ [29,000 ksi]); Both the raw and decomposed data are presented as response history, and temperature versus gauge reading (either stress, displacement, or rotation) formats.

The location of each instrumented cross-section, tiltmeter, and crackmeter for each bridge, can be observed in Figures 1 and 2. Besides, the number of gauges and their location are presented in Volume I (Zhou et al. 2026), which is better understood when looked at in conjunction with the girder cross-section and cross-frame representation shown in Figure 3.

IDENTIFICATION AND HANDLING OF SPURIOUS STRAIN GAUGE READINGS

Several of the readings of gauges presented in this appendix have been trimmed before the end of the monitoring period. There are two main reasons why a gauge might not have readings spanning the full monitoring period: 1) The gauge stopped working (i.e., no further readings were collected from the gauge), or 2) the gauge readings were deemed as spurious.

Over the monitoring period, several of the installed gauges were observed to have significant deviations from typical readings that were deemed by the research team as spurious. An example of spurious readings is shown in Figure 78 and Figure 79 for Mattis-57 and Mattis-74, respectively. Although this behavior was initially considered as real bridge behavior, further investigation exposed gauge malfunction. Fully instrumented cross-sections—where gauges were installed at four locations on the bottom flange (BWt, BWb, BEt, and BEb in Figure 3-A)—were particularly helpful to identify spurious behavior. In Figure 78 for example, one can observe that while the BEb gauge deviates, there is no alteration on the gauge readings mounted on the top side of the same flange (BEt). A similar scenario is observed on Figure 79, where the BWt gauge deviates significantly while the BWb gauge remains within typical ranges. It is observed that in many cases, deviations happen in the colder months, typically around January, although a few deviations also occur during warmer months. It was also observed that many of the deviations happened in tension (increasing strain), and only a few presented a reversal in strain after deviations happened, while most of them continued deviating over time.

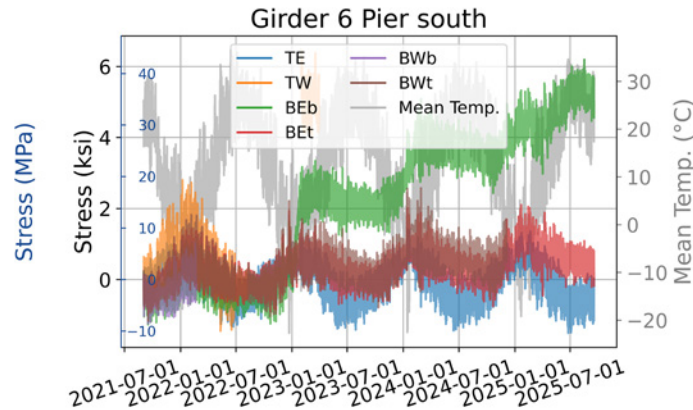


Figure 78. Graph. Example of spurious readings on Mattis-57.

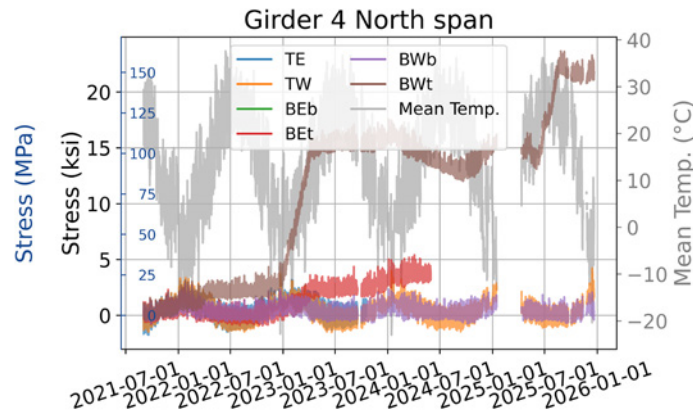
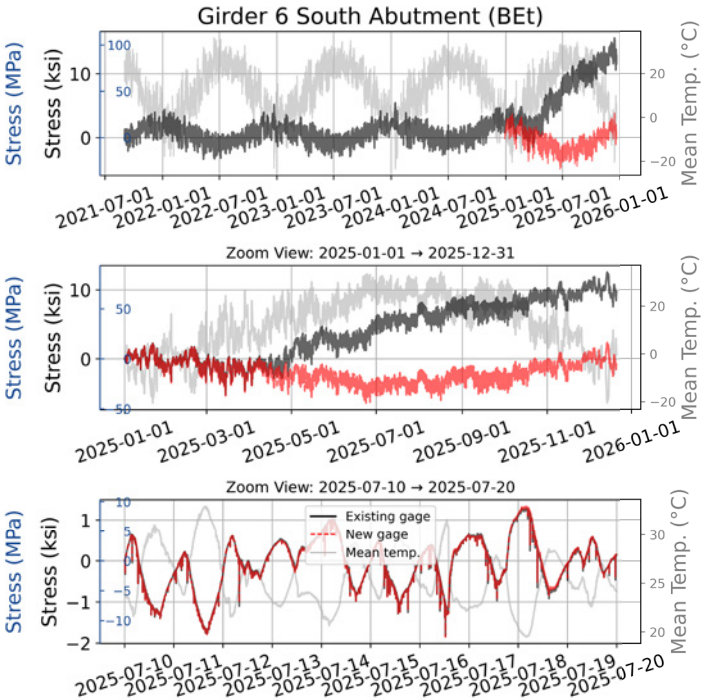


Figure 79. Graph. Example of spurious readings on Mattis-74.

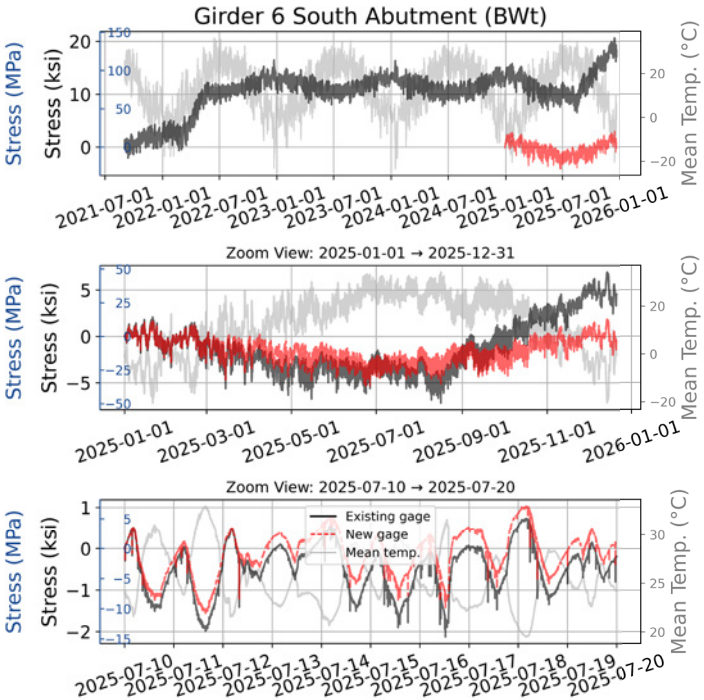
After communication with the Geokon (Geokon, Inc. 2022) team (gauge manufacturer), no clear reason for this malfunction was found. The gauges were inspected in the field, and no clear sign of corrosion or weld issues was observed.

To confirm the strain gauge malfunctions, 4 additional strain gauges were installed on Mattis-57 bridge. These new gauges were installed adjacent to the existing gauges that presented deviation in readings, with a free spacing of 13 cm (5 in.) between gauges (in the bridge longitudinal direction). The readings from these new gauges, together with the readings from the existing gauges adjacent to them are shown in Figure 80. Three plots per gauge are presented: the top plot shows the new gauge together with the full response history of the existing gauge, the middle plot shows a zoomed-in view of only the period of time when the new gauge was installed, and the bottom plot shows both the existing and new gauge readings over a period of 10 days during the summer of 2025. The readings are reset to zero at the beginning of each plot. It is observed that, for all four gauges, the short-term behavior (bottom plot) is almost identical. In the long-term (middle and upper plots), new gauges show a behavior that can be categorized as typical for each location, whereas the existing gauge show signs of spurious readings (deviation from typical behavior), which are particularly evident for the BEt and BWt gauges on the girder 6 south abutment cross-section.

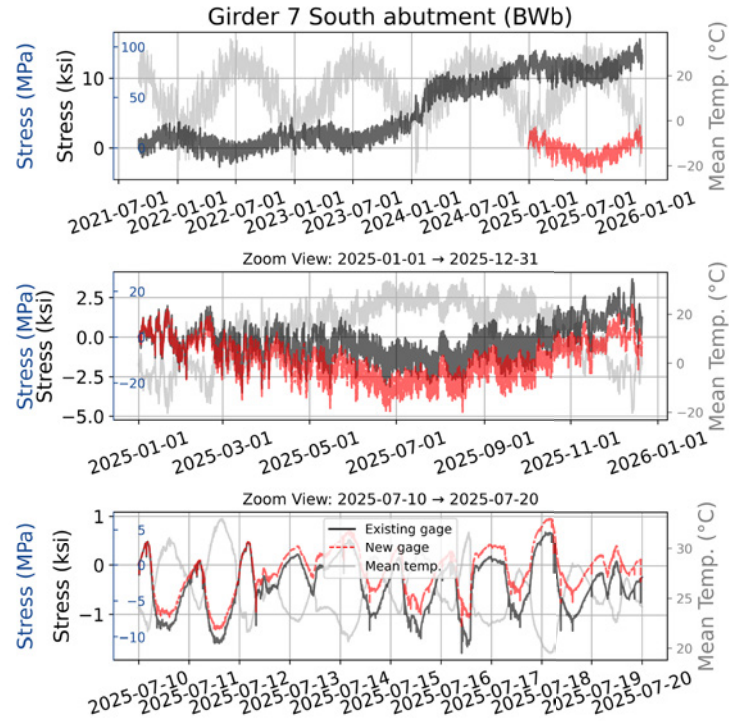
It is based on this assessment, together with the discussion on Figure 78 and Figure 79, that readings after observed deviation were discarded for the data shown in the following sections, and for any conclusions drawn in this report.



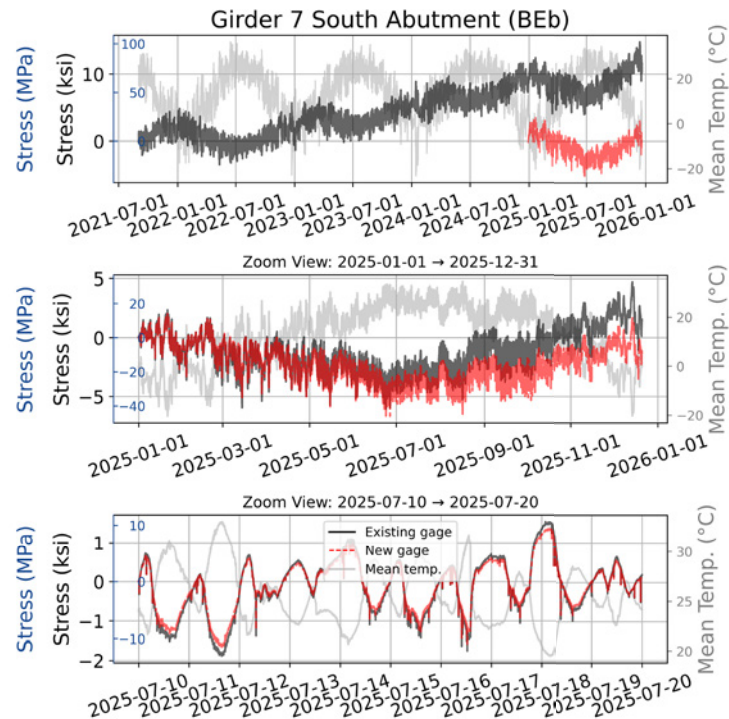
A. Girder 6 South Abutment (BEt)



B. Girder 6 South Abutment (BWt)



C. Girder 7 South Abutment (BWb)



D. Girder 7 South Abutment (BEb)

Figure 80. Graph. Additional gauges installed on Mattis-57.

DATA COLLECTED FROM MATTIS-74 (STUB)

Stress Data from Girder Cross-Sections

A total of 10 girder cross-sections were instrumented in the Mattis-74 bridge, 4 of which were heavily instrumented, as described in Chapter 1. The total number of strain gauges mounted on girder cross-sections is 60 (56 mounted on steel I-girders and 4 embedded in the concrete deck).

To provide an overview of the recorded response, Table 22 summarizes the key measures for all gauges, including maximum, minimum, and overall range of stress values over the monitoring period.

Table 22. Summary of Stress Data from Girder Cross-Section Gauges—Mattis-74

Cross Section	Gauge	Min. Stress MPa (ksi)	Temp. at Min. (°C)	Max. Stress MPa (ksi)	Temp. at Max. (°C)	Stress Range MPa (ksi)
G1 N. Span	BEb	−5.5 (−0.8)	34.1	26.5 (3.8)	24.7	32 (4.6)
G1 N. Span	BWb	−9 (−1.3)	25.2	38.5 (5.6)	24.7	47.5 (6.9)
G1 N. Span	TE	−9 (−1.3)	18.0	28.5 (4.1)	−3.3	37.5 (5.4)
G1 N. Span	TW	−10 (−1.4)	11.9	21.5 (3.1)	15.5	31.5 (4.6)
G1 Pier	BEb	−10 (−1.5)	33.8	28 (4.1)	−15.8	38 (5.5)
G1 Pier	TW	−6 (−0.9)	17.4	13.5 (1.9)	2.5	19.5 (2.8)
G3 N. Span	BEb	−4.5 (−0.6)	25.9	19.5 (2.8)	−3.4	23.5 (3.4)
G3 N. Span	BWb	−3.5 (−0.5)	30.6	22.5 (3.2)	2.7	26 (3.8)
G3 N. Span	TW	−9 (−1.3)	24.2	27.5 (4)	−22.4	36.5 (5.3)
G3 Pier	BEb	−8.5 (−1.2)	30.6	31.5 (4.5)	−21.5	40 (5.8)
G3 Pier	TE	−14.5 (−2.1)	27.9	5 (0.7)	2.4	19.5 (2.8)
G3 Pier	TW	−11.5 (−1.6)	28.2	10.5 (1.5)	−16.7	22 (3.2)
G4 N. Span	BEt	−6.5 (−0.9)	21.9	18.5 (2.7)	−13.3	24.5 (3.6)
G4 N. Span	BWb	−5.5 (−0.8)	29.7	20 (2.9)	−13.3	25 (3.7)
G4 N. Span	TE	−12 (−1.8)	28.1	24.5 (3.6)	−22.7	37 (5.3)
G4 N. Span	TW	−10.5 (−1.5)	26.8	29 (4.2)	−17.3	39.5 (5.7)
G4 Pier	BEb	−9 (−1.3)	21.0	17 (2.5)	−15.9	26 (3.8)
G4 Pier	BEt	−7 (−1)	29.4	19 (2.7)	−15.9	26 (3.8)
G4 Pier	BWb	−7.5 (−1.1)	23.0	13 (1.9)	−15.6	20.5 (2.9)
G4 Pier	BWt	−7.5 (−1.1)	23.0	9 (1.3)	4.3	16.5 (2.4)
G4 Pier	TE	−9.5 (−1.3)	30.9	19 (2.8)	−16.3	28.5 (4.1)
G4 Pier	TW	−9 (−1.3)	20.9	10.5 (1.5)	−10.5	19 (2.8)
G4 S. Span	TW	−7 (−1)	28.1	29.5 (4.3)	−22.1	36.5 (5.3)
G5 Pier	BEb	−10.5 (−1.5)	28.4	16 (2.3)	−15.7	26.5 (3.9)
G5 Pier	BEt	−7.5 (−1.1)	24.6	4 (0.5)	4.2	11.5 (1.7)
G5 Pier	BWb	−8.5 (−1.2)	22.8	20 (2.9)	−18.3	28.5 (4.1)
G5 Pier	BWt	−8 (−1.2)	22.8	10.5 (1.5)	−15.4	18.5 (2.7)
G5 Pier	TE	−7 (−1)	8.6	23 (3.3)	−16.6	30 (4.3)

Cross Section	Gauge	Min. Stress MPa (ksi)	Temp. at Min. (°C)	Max. Stress MPa (ksi)	Temp. at Max. (°C)	Stress Range MPa (ksi)
G5 Pier	TW	-10.5 (-1.5)	29.2	18 (2.6)	-17.2	28.5 (4.1)
G5 S. Span	BEb	-8 (-1.1)	24.9	15 (2.2)	-0.3	23 (3.3)
G5 S. Span	BWb	-10 (-1.4)	15.2	19 (2.8)	-17.9	29 (4.2)
G5 S. Span	TE	-8.5 (-1.2)	32.1	27.5 (4)	-18.0	36 (5.2)
G5 S. Span	TW	-9.5 (-1.4)	32.2	31.5 (4.5)	-18.1	40.5 (5.9)
G5 N. Span	BEb	-5 (-0.7)	26.8	20 (2.9)	-13.5	25 (3.6)
G5 N. Span	BEt	-5 (-0.8)	26.8	20 (2.9)	-13.5	25 (3.7)
G5 N. Span	BWb	-6 (-0.8)	15.6	18.5 (2.7)	-13.5	24.5 (3.6)
G5 N. Span	BWt	-4.5 (-0.6)	26.8	20 (2.9)	-13.5	24.5 (3.6)
G5 N. Span	TW	-12 (-1.8)	27.3	25.5 (3.7)	-21.9	38 (5.5)

All data corresponding to the summary presented in Table 22 is illustrated in the response history plots below (Figure 81 through Figure 90), which show the full temporal evolution of the recorded stresses.

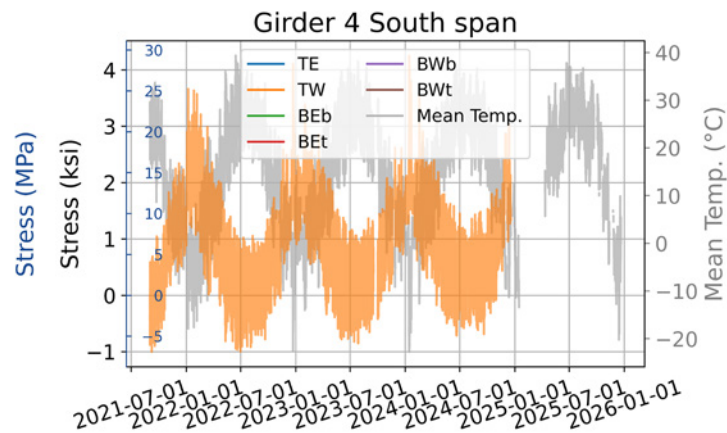


Figure 81. Graph. Stress data collected from Girder 4 South Span.

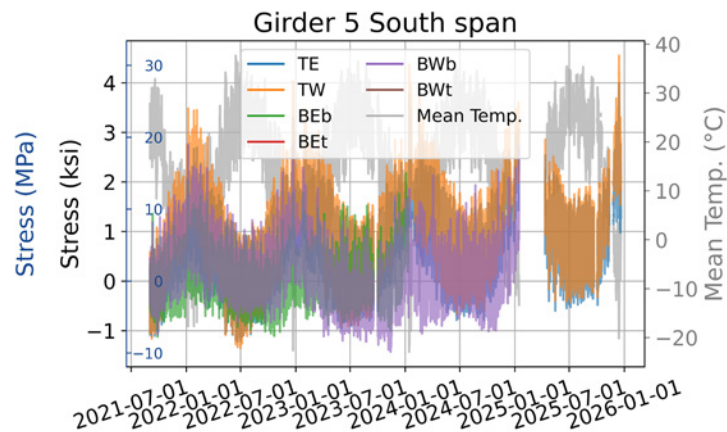


Figure 82. Graph. Stress data collected from Girder 5 South Span.

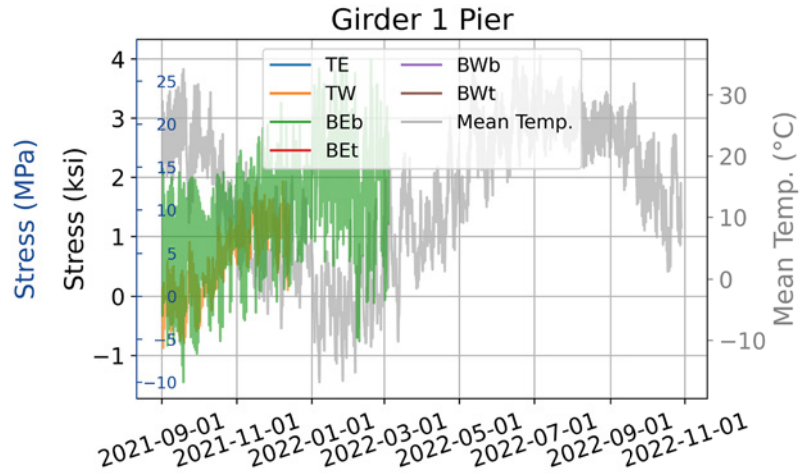


Figure 83. Graph. Stress data collected from Girder 1 Pier.

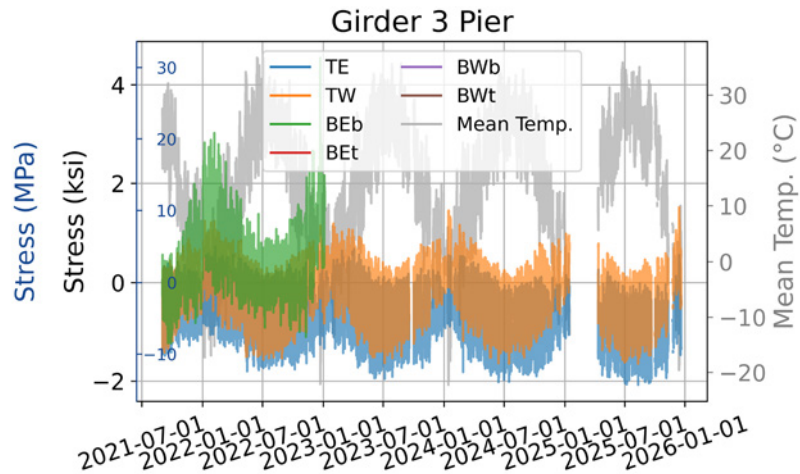


Figure 84. Graph. Stress data collected from Girder 3 Pier.

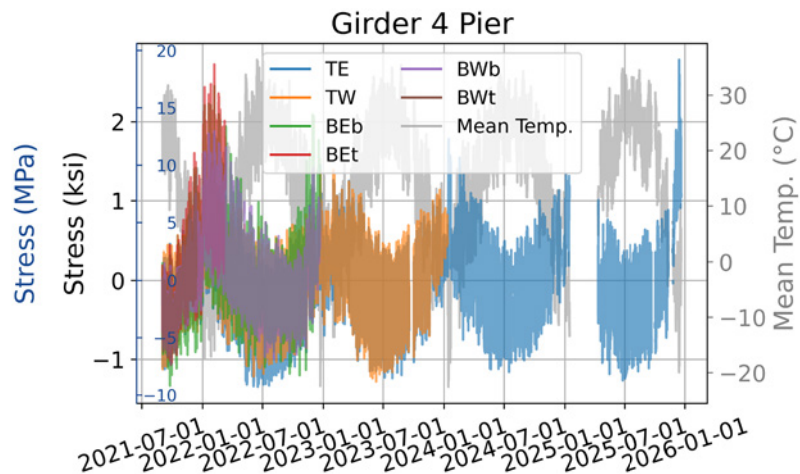


Figure 85. Graph. Stress data collected from Girder 4 Pier.

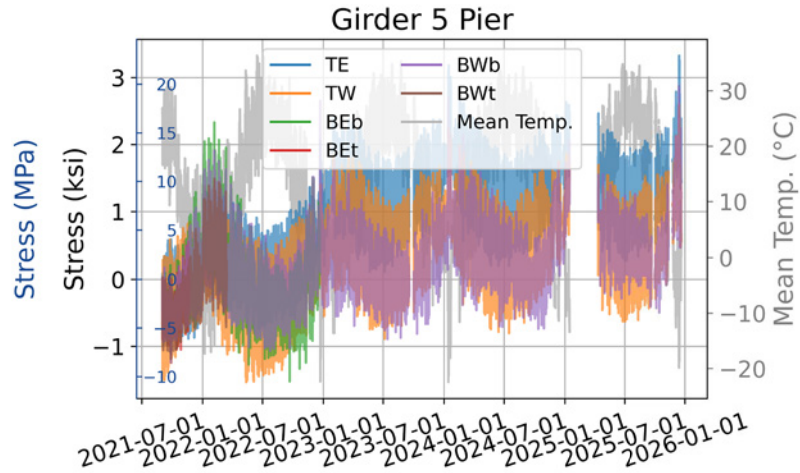


Figure 86. Graph. Stress data collected from Girder 5 Pier.

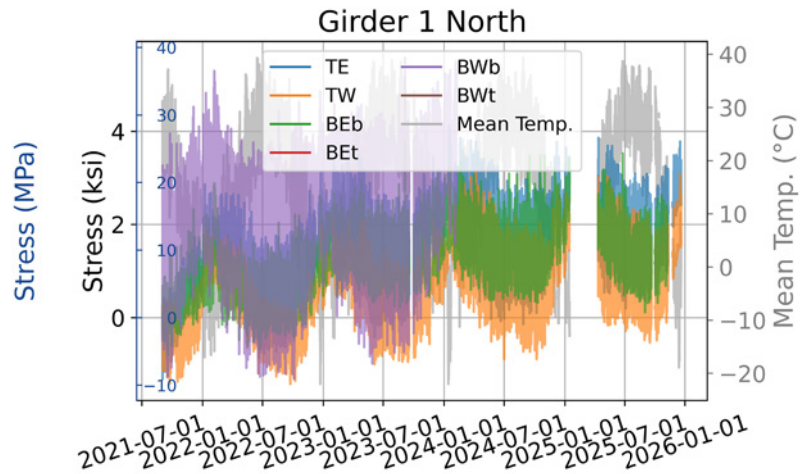


Figure 87. Graph. Stress data collected from Girder 1 North Span.

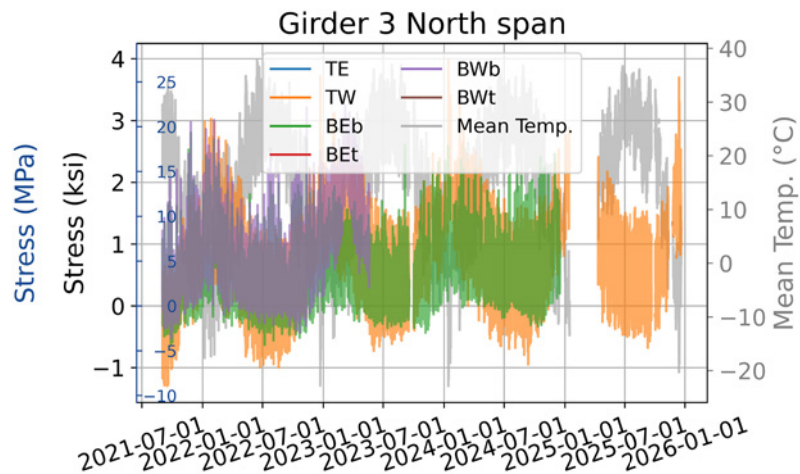


Figure 88. Graph. Stress data collected from Girder 3 North Span.

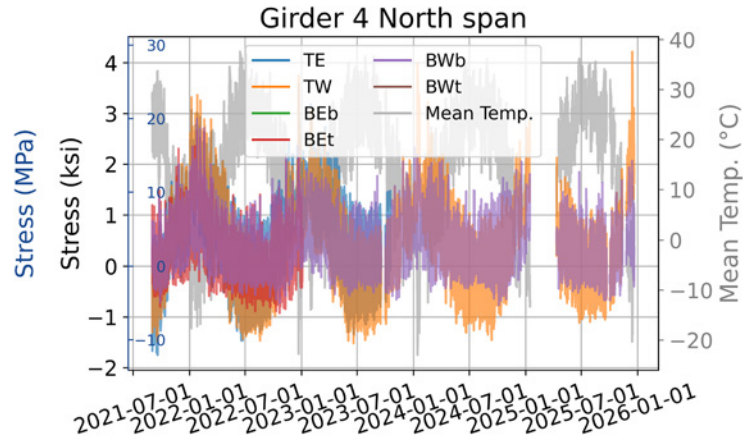


Figure 89. Graph. Stress data collected from Girder 4 North Span.

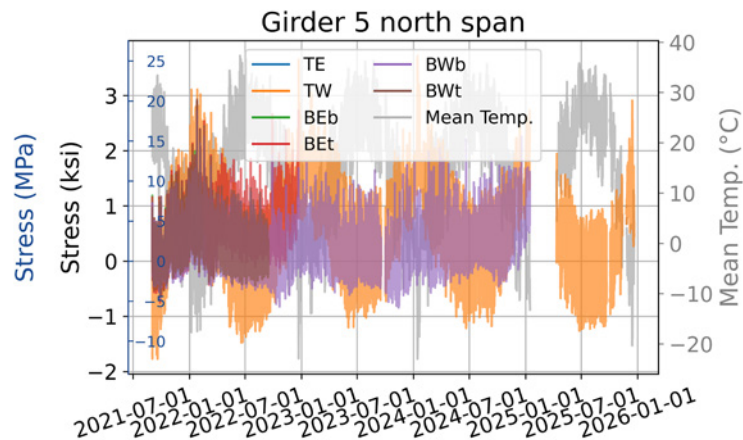


Figure 90. Graph. Stress data collected from Girder 5 North Span.

The same data presented above, is presented in temperature versus stress readings in the plots below (Figure 91 through Figure 100). The temperature used for these plots is the average temperature collected from thermistors attached to all gauges mounted on the cross-section.

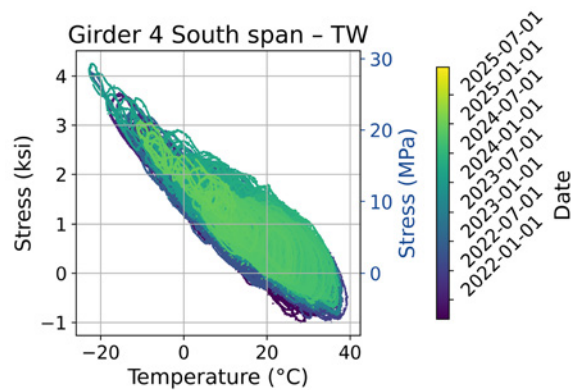


Figure 91. Graph. Temperature vs. stress data collected from Girder 4 South Span.

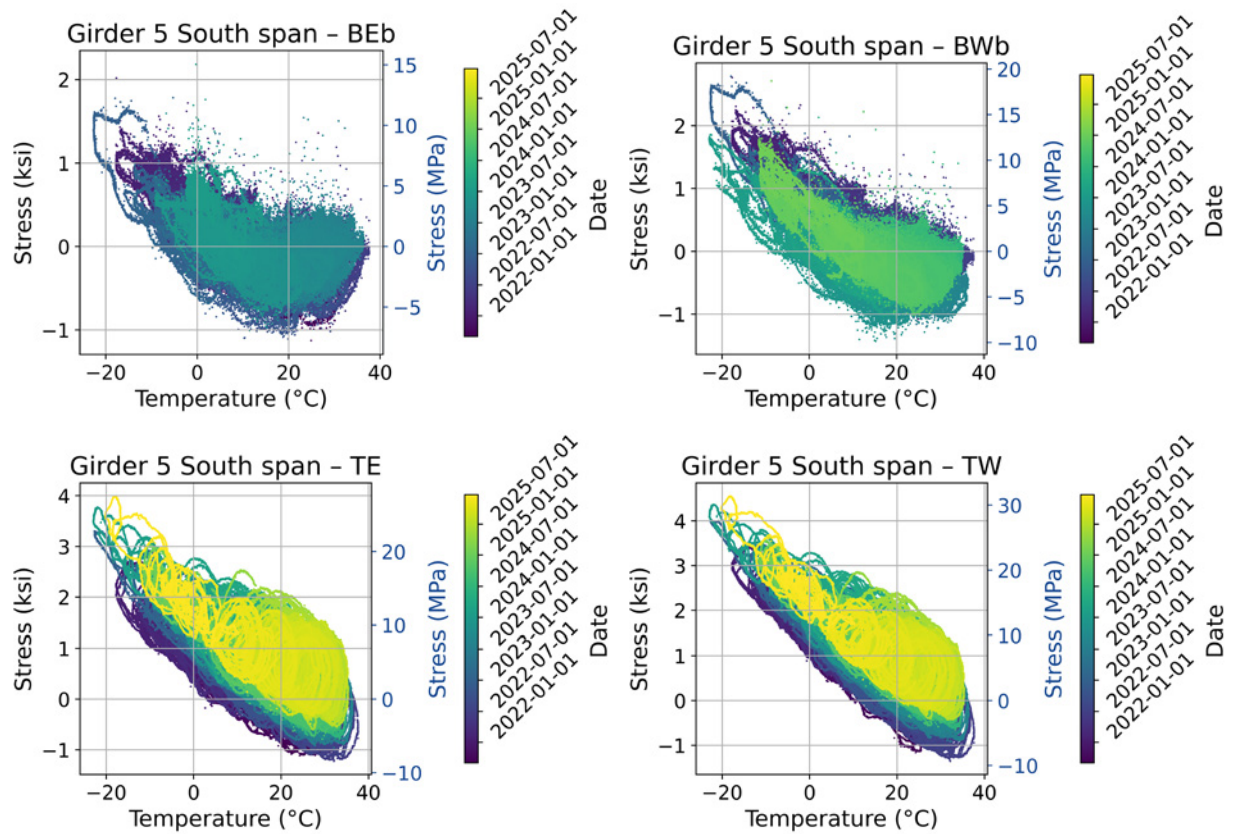


Figure 92. Graph. Temperature vs. stress data collected from Girder 5 South Span.

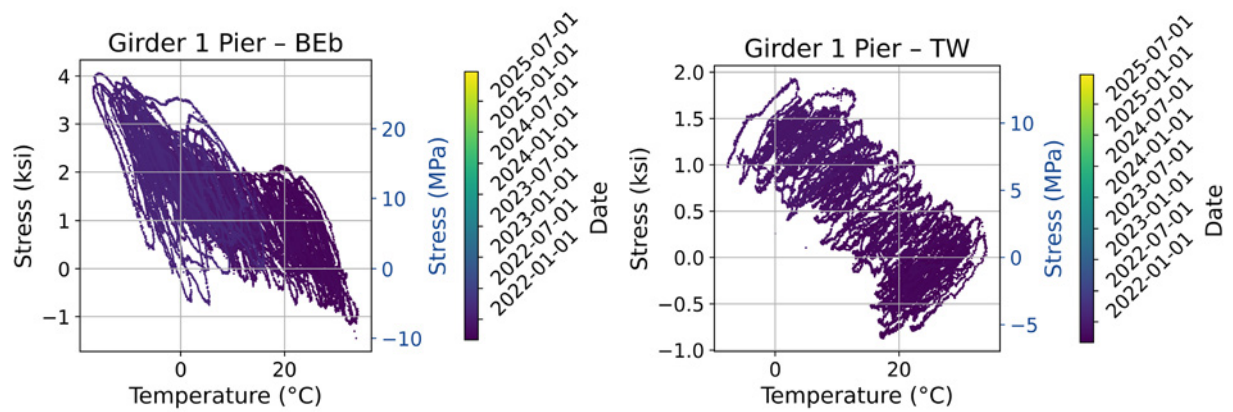


Figure 93. Graph. Temperature vs. stress data collected from Girder 1 Pier.

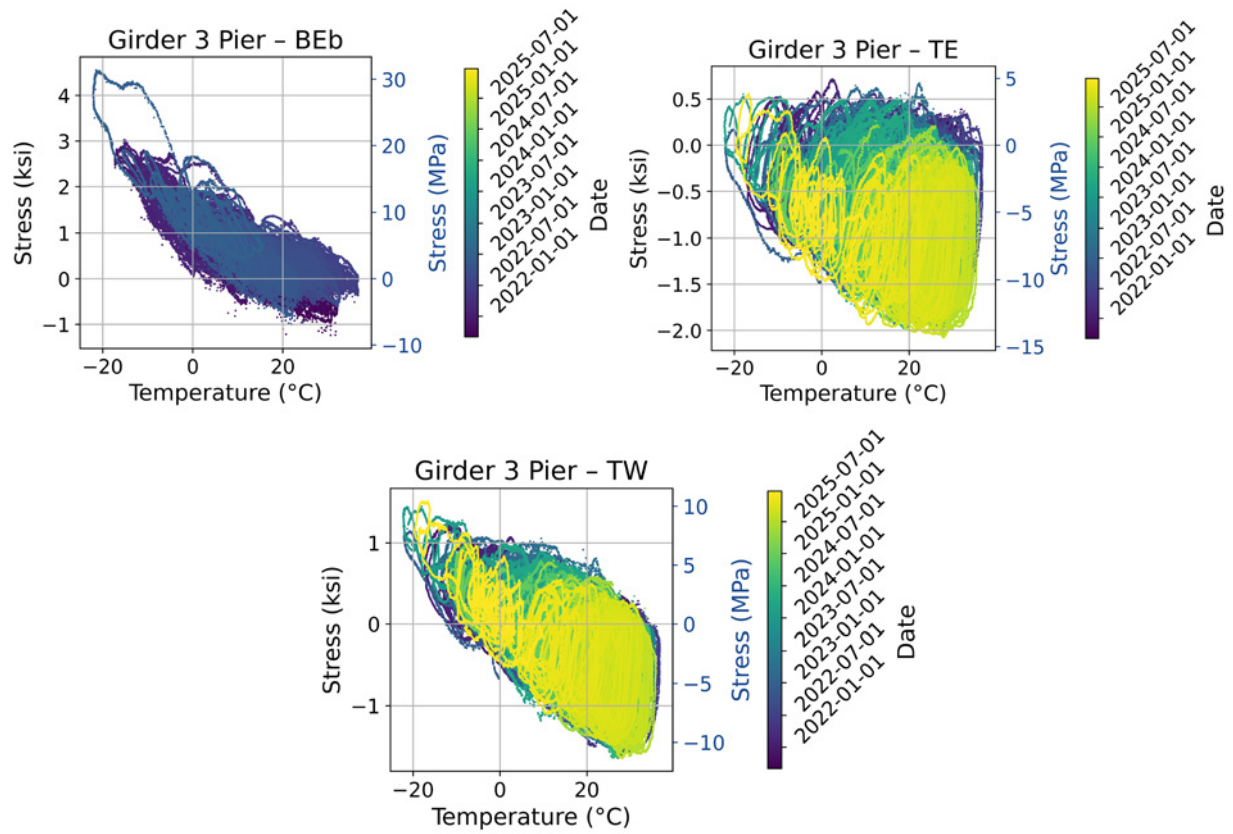


Figure 94. Graph. Temperature vs. stress data collected from Girder 3 Pier.

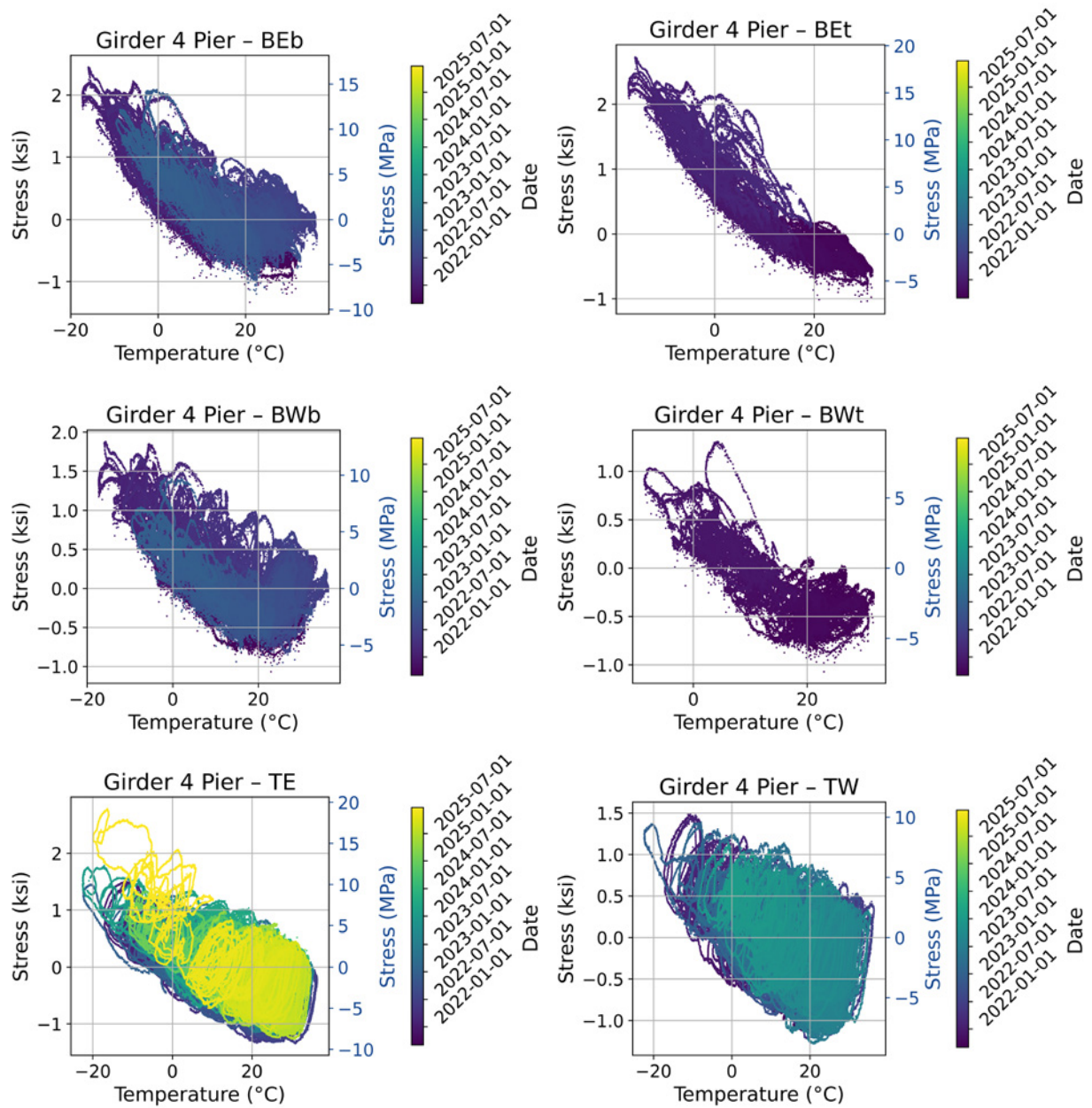


Figure 95. Graph. Temperature vs. stress data collected from Girder 4 Pier.

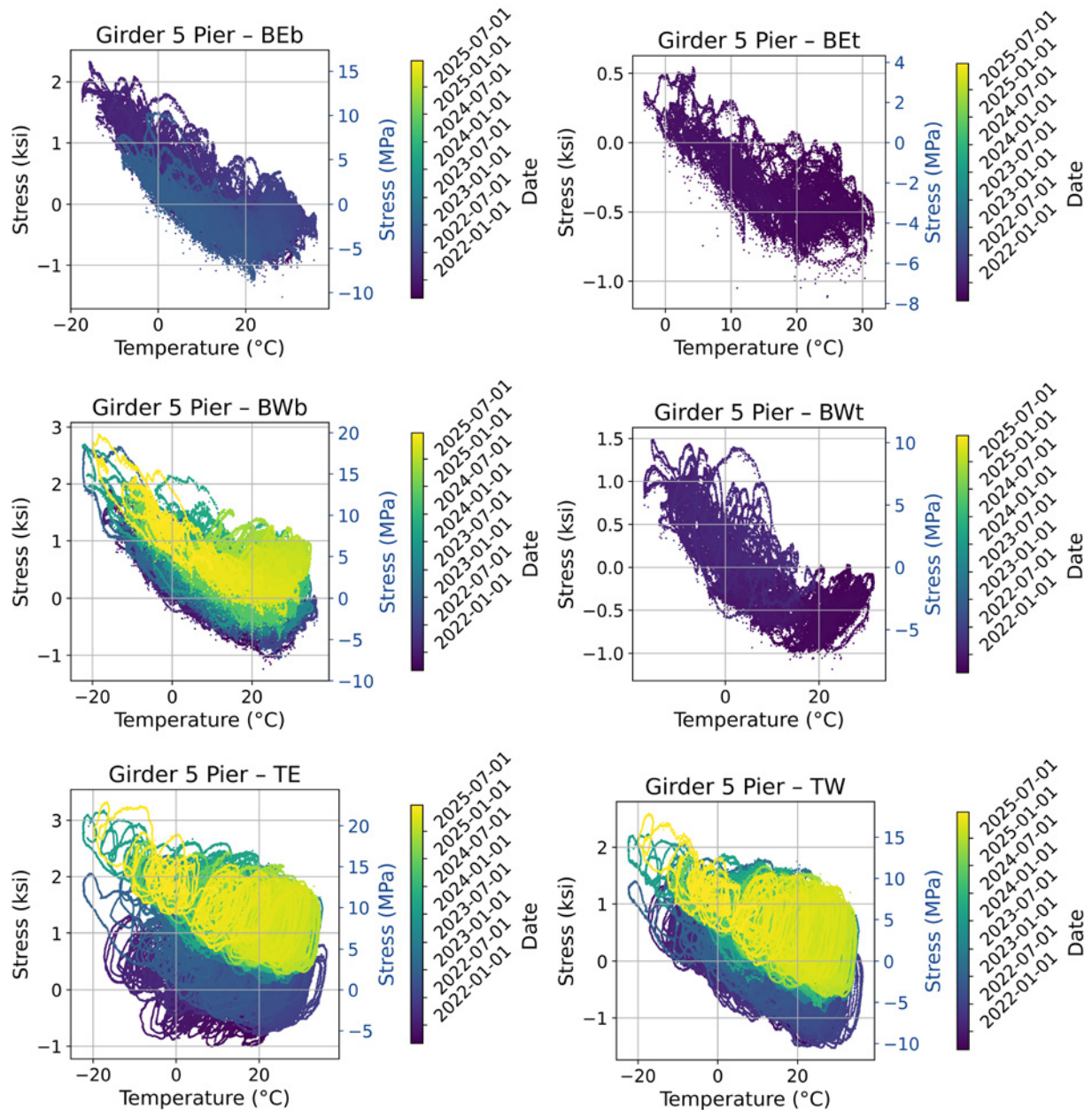


Figure 96. Graph. Temperature vs. stress data collected from Girder 5 Pier.

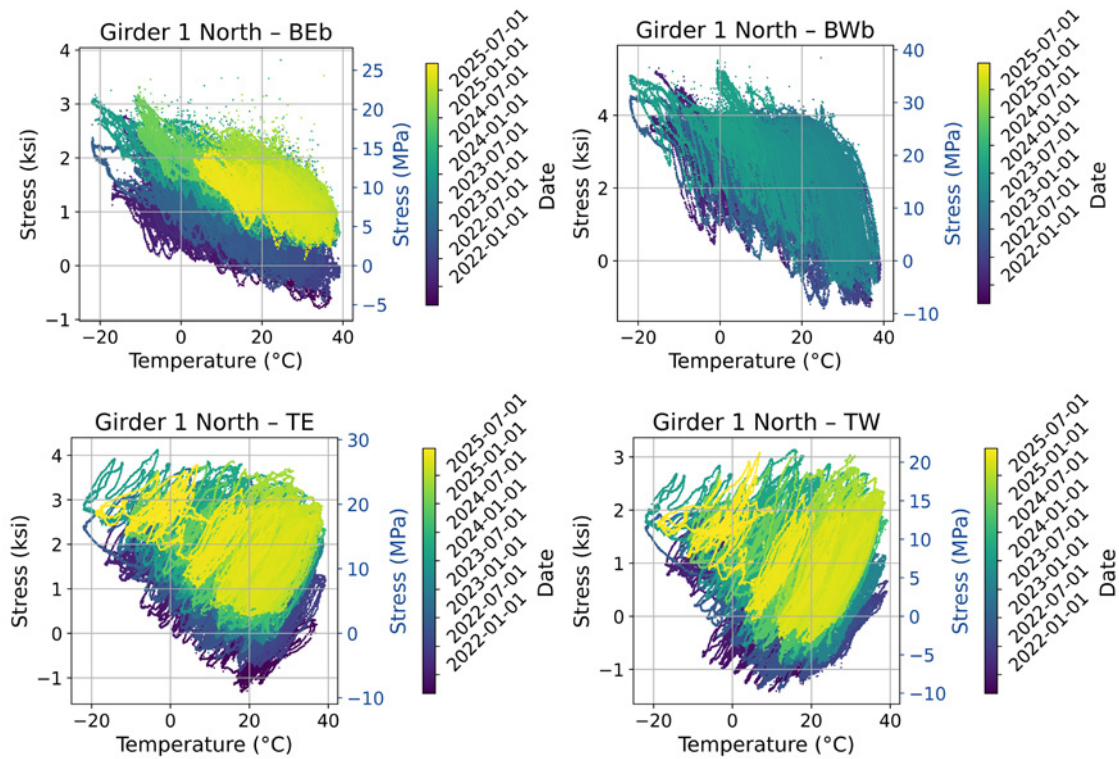


Figure 97. Graph. Temperature vs. stress data collected from Girder 1 North Span.

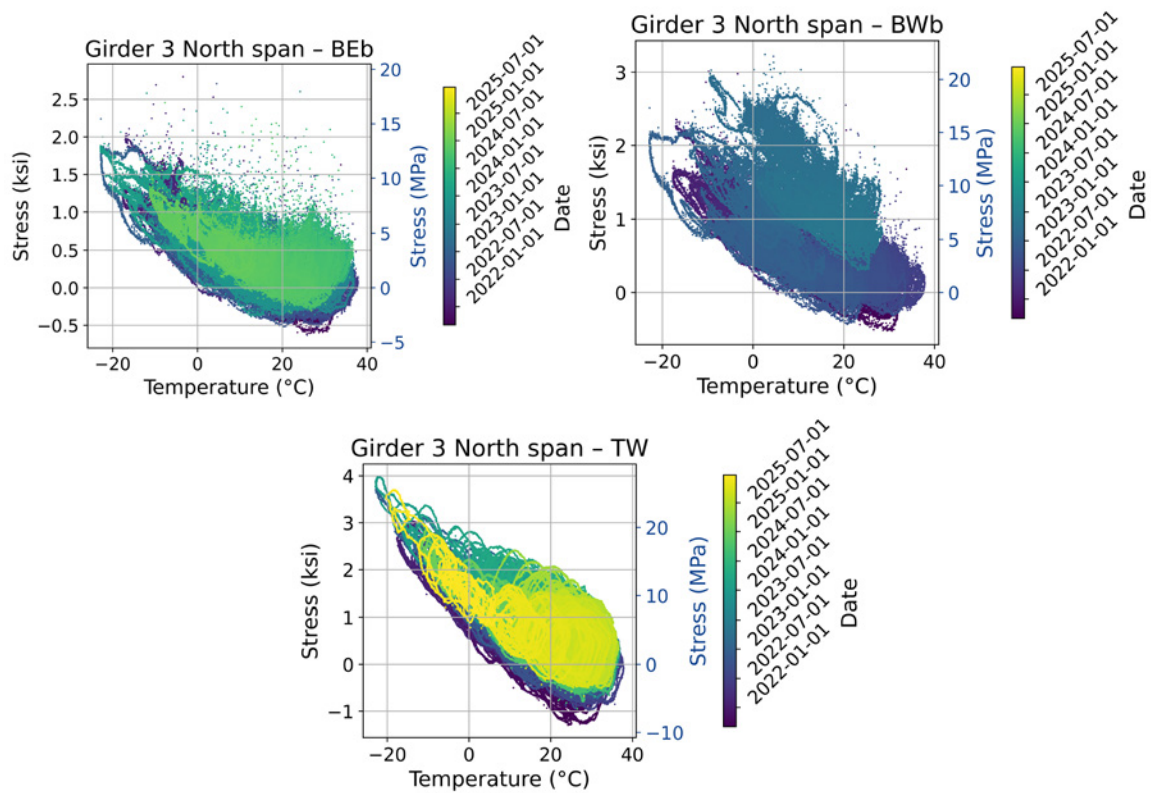


Figure 98. Graph. Temperature vs. stress data collected from Girder 3 North Span.

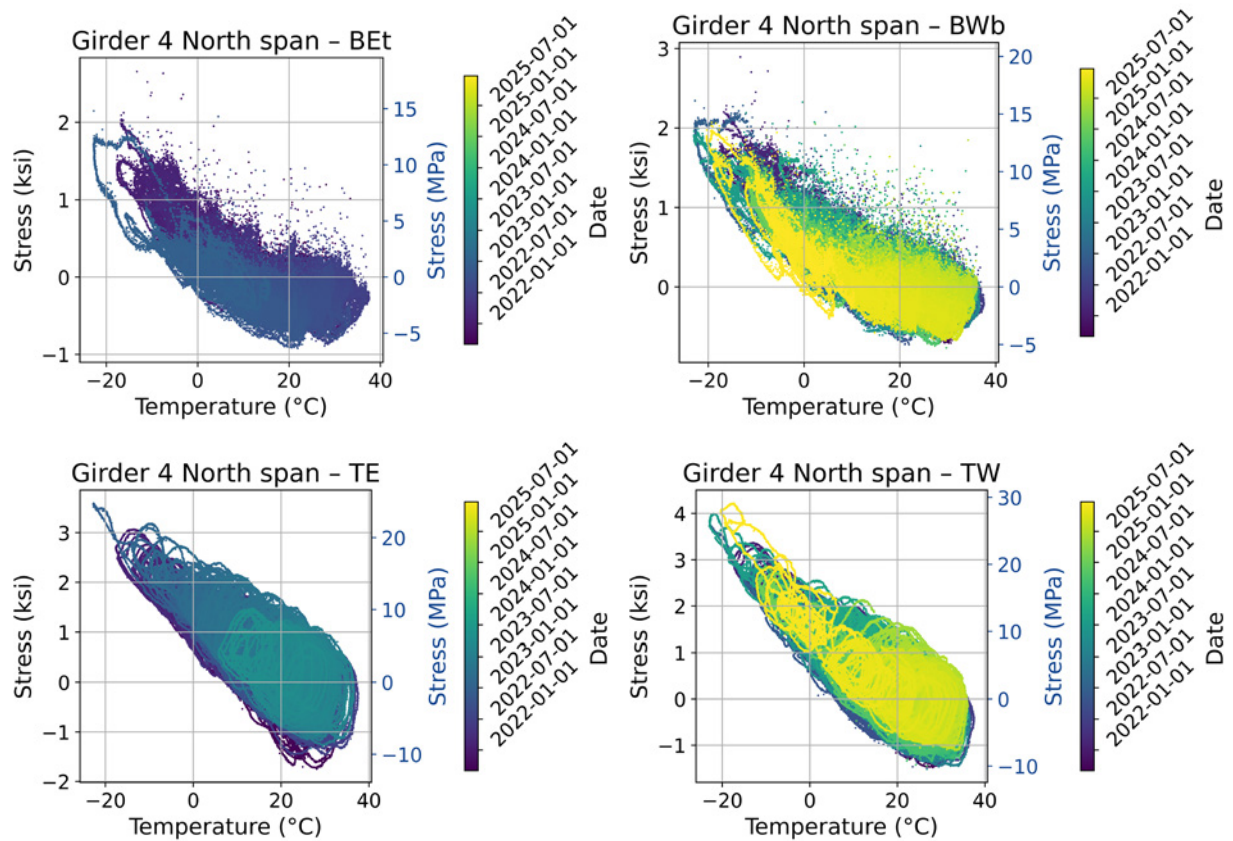


Figure 99. Graph. Temperature vs. stress data collected from Girder 4 North Span.

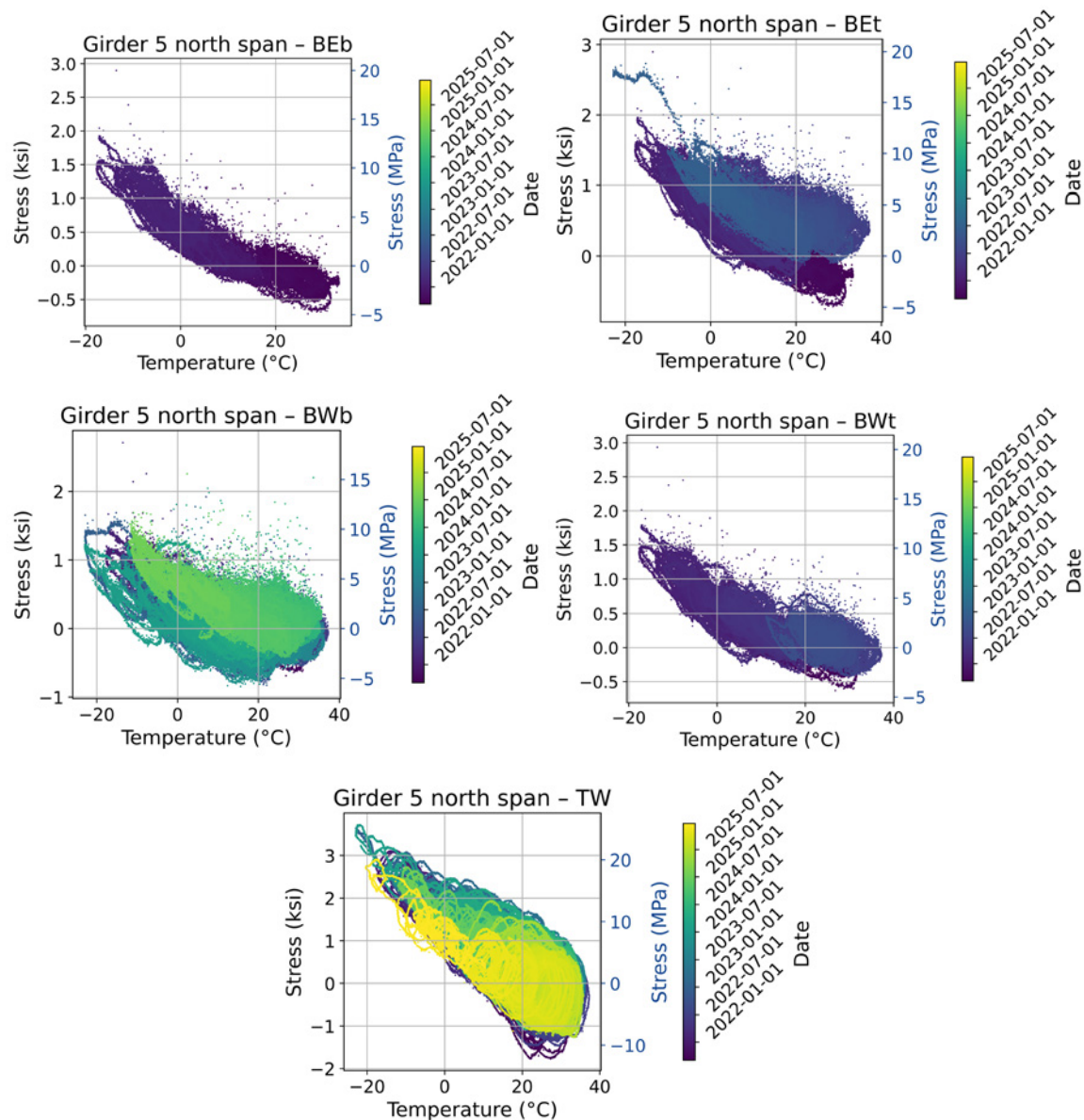


Figure 100. Graph. Temperature vs. stress data collected from Girder 5 North Span.

Decomposed Data from Girder Cross-Sections

Stress data obtained from girder cross-sections was decomposed into individual girder actions, including strong-axis bending, weak-axis bending, and axial stress, following the scheme presented in Volume I (Zhou et al. 2026b).

To provide an overview of these response components, Table 23 summarizes the key measures for all cross-sections, including maximum, minimum, and overall range of values over the monitoring period.

Table 23. Summary of Decomposed Stress Components from Girder Cross-Section Gauges—Mattis-74

Cross Section	Component	Min. Stress MPa (ksi)	Temp. at Min. (°C)	Max. Stress MPa (ksi)	Temp. at Max. (°C)	Stress Range MPa (ksi)
G1 N. Span	Axial	-5 (-0.7)	18.0	22 (3.2)	-14.1	27 (3.9)
G1 N. Span	SA bottom	-14 (-2.1)	26.5	28.5 (4.1)	24.7	42.5 (6.2)
G1 N. Span	SA top	-7 (-1)	24.7	3.5 (0.5)	26.5	10.5 (1.5)
G1 N. Span	WA bottom	-11.5 (-1.7)	25.6	5.5 (0.8)	36.3	17 (2.4)
G1 N. Span	WA top	-1.5 (-0.2)	32.0	6.5 (1)	23.8	8 (1.2)
G4 N. Span	Axial	-9 (-1.3)	27.8	23 (3.3)	-22.7	32 (4.6)
G4 N. Span	SA bottom	-12.5 (-1.8)	-20.4	15 (2.2)	23.5	27.5 (4)
G4 N. Span	SA top	-4 (-0.5)	23.5	3 (0.5)	-20.4	7 (1)
G4 N. Span	WA bottom	-2 (-0.3)	18.2	1.5 (0.2)	33.4	3.5 (0.5)
G4 N. Span	WA top	-2.5 (-0.4)	2.0	2.5 (0.3)	14.5	4.5 (0.7)
G4 Pier	Axial	-6.5 (-0.9)	25.4	10 (1.5)	-11.8	16.5 (2.4)
G4 Pier	SA bottom	-8.5 (-1.3)	21.0	11 (1.6)	-1.6	20 (2.9)
G4 Pier	SA top	-4 (-0.6)	-1.6	3 (0.4)	21.0	7 (1)
G4 Pier	WA bottom	-3 (-0.4)	20.3	3 (0.4)	15.1	5.5 (0.8)
G4 Pier	WA top	-2.5 (-0.3)	35.4	1.5 (0.2)	-8.6	3.5 (0.5)
G5 Pier	Axial	-6.5 (-0.9)	20.1	10 (1.4)	-12.0	16.5 (2.4)
G5 Pier	SA bottom	-10.5 (-1.5)	20.5	10.5 (1.5)	2.0	20.5 (3)
G5 Pier	SA top	-3.5 (-0.5)	2.0	3.5 (0.5)	20.5	7 (1)
G5 Pier	WA bottom	-2 (-0.3)	21.9	2 (0.3)	-16.5	4 (0.6)
G5 Pier	WA top	-3 (-0.4)	0.2	4.5 (0.7)	31.8	7.5 (1.1)
G5 S. Span	Axial	-7 (-1)	34.0	23 (3.3)	-22.7	30 (4.3)
G5 S. Span	SA bottom	-18 (-2.6)	-0.9	12 (1.8)	30.5	29.5 (4.3)
G5 S. Span	SA top	-3 (-0.4)	30.5	4 (0.6)	-0.9	7 (1)
G5 S. Span	WA bottom	-3.5 (-0.5)	-21.9	3 (0.5)	14.1	7 (1)
G5 S. Span	WA top	-4 (-0.6)	2.7	1 (0.2)	29.0	5.5 (0.8)

All data corresponding to the summary presented in Table 23 is illustrated in the response history plots below (Figure 101 through Figure 105), which shows the full temporal evolution of each decomposed stress component per girder cross-section.

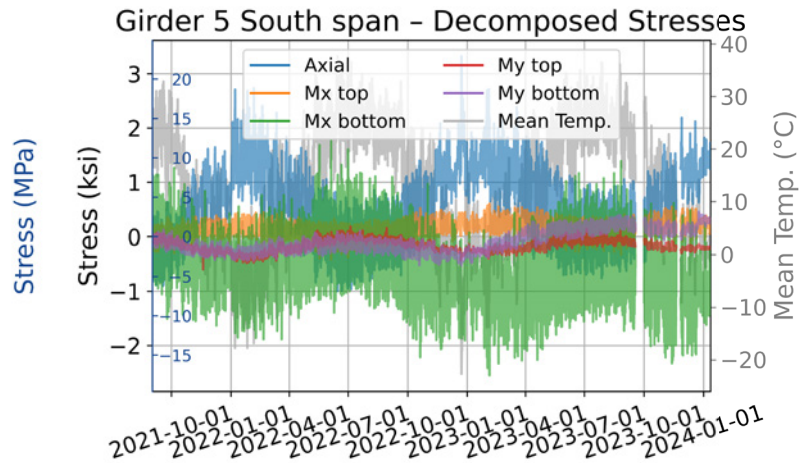


Figure 101. Graph. Decomposed stress data collected from Girder 5 South Span.

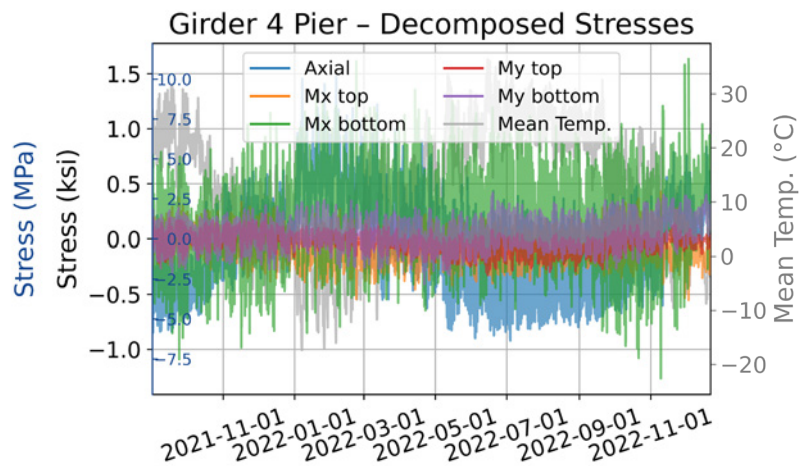


Figure 102. Graph. Decomposed stress data collected from Girder 4 Pier.

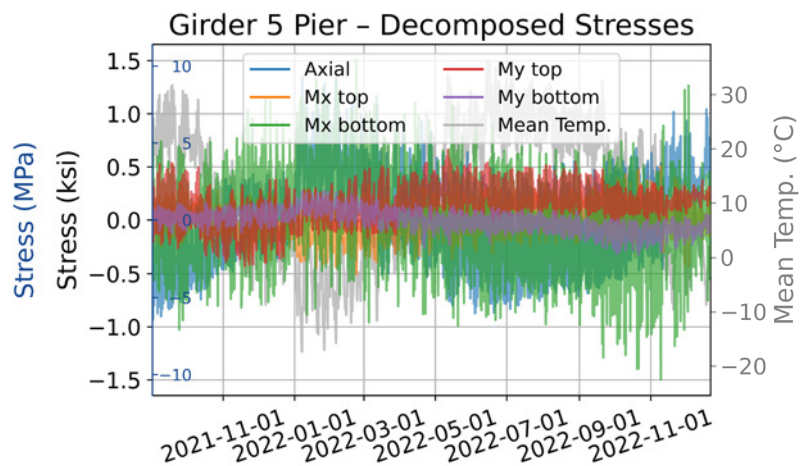


Figure 103. Graph. Decomposed stress data collected from Girder 5 Pier.

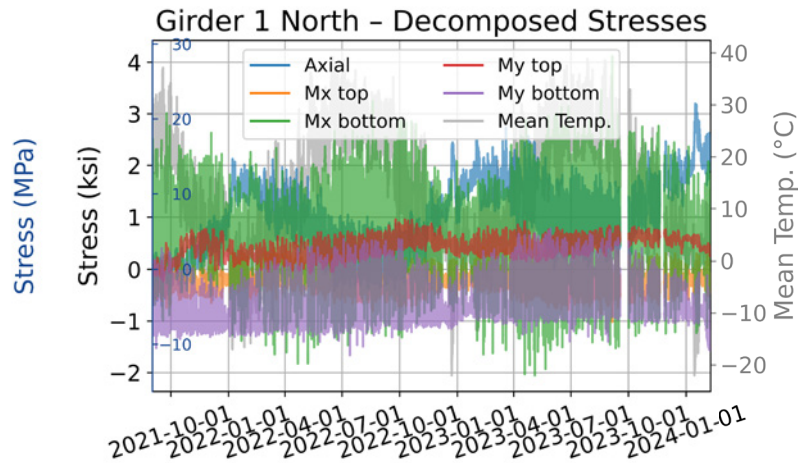


Figure 104. Graph. Decomposed stress data collected from Girder 1 North Span.

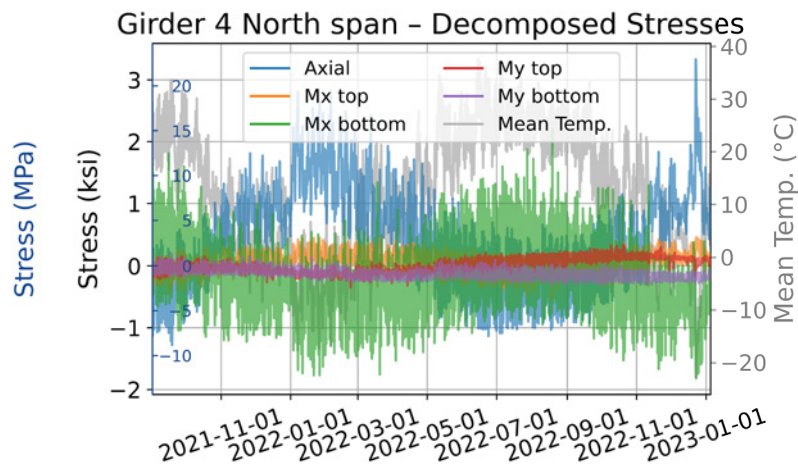


Figure 105. Graph. Decomposed stress data collected from Girder 4 North Span.

The same decomposed stress data presented above is presented in temperature versus decomposed stress readings in the plots below (Figure 106 through Figure 110). The temperature used for these plots is the average temperature collected from thermistors attached to all gauges mounted on the cross-section.

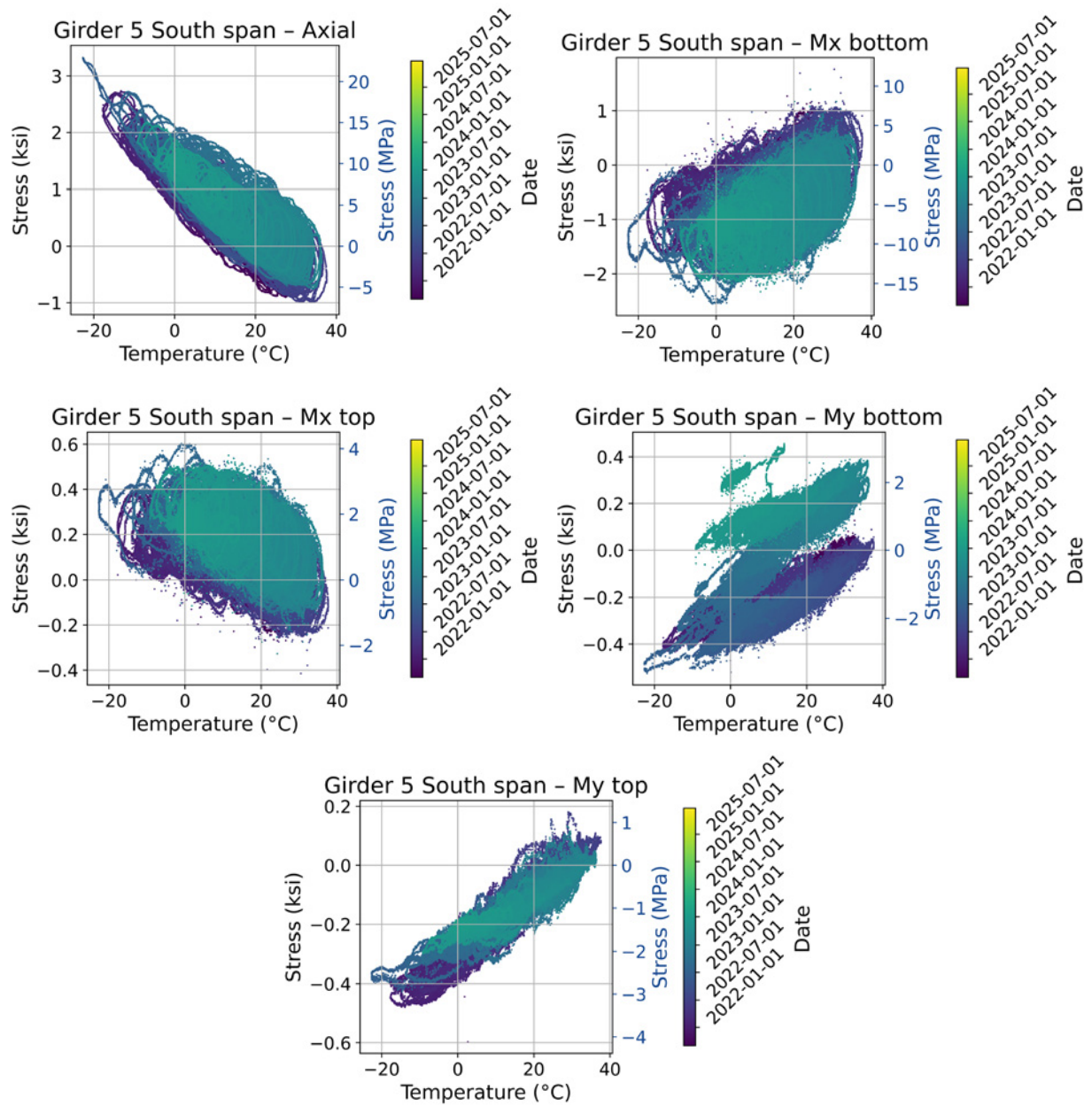


Figure 106. Graph. Temperature vs. decomposed stress data collected from Girder 5 South Span.

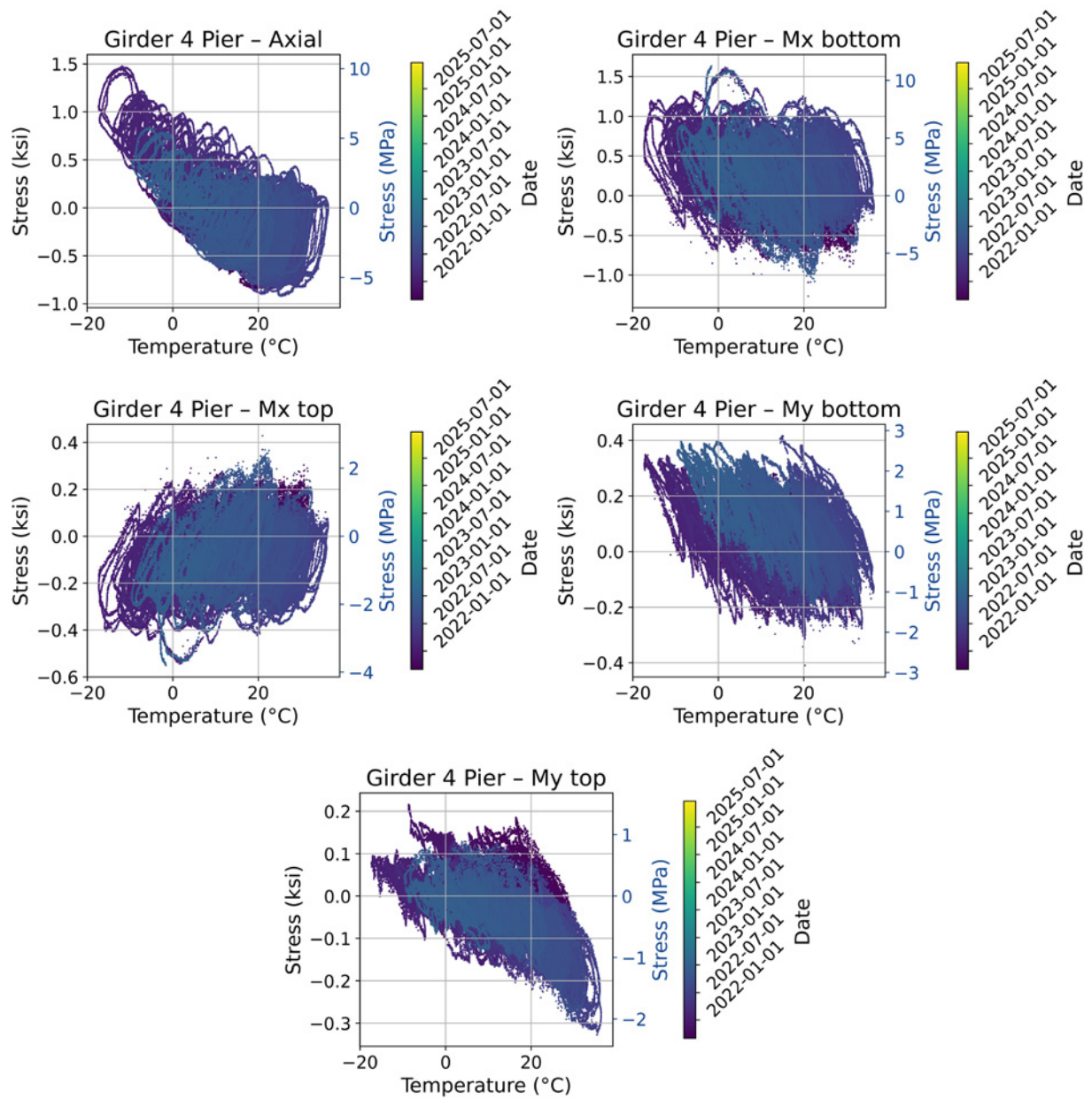


Figure 107. Graph. Temperature vs. decomposed stress data collected from Girder 4 Pier.

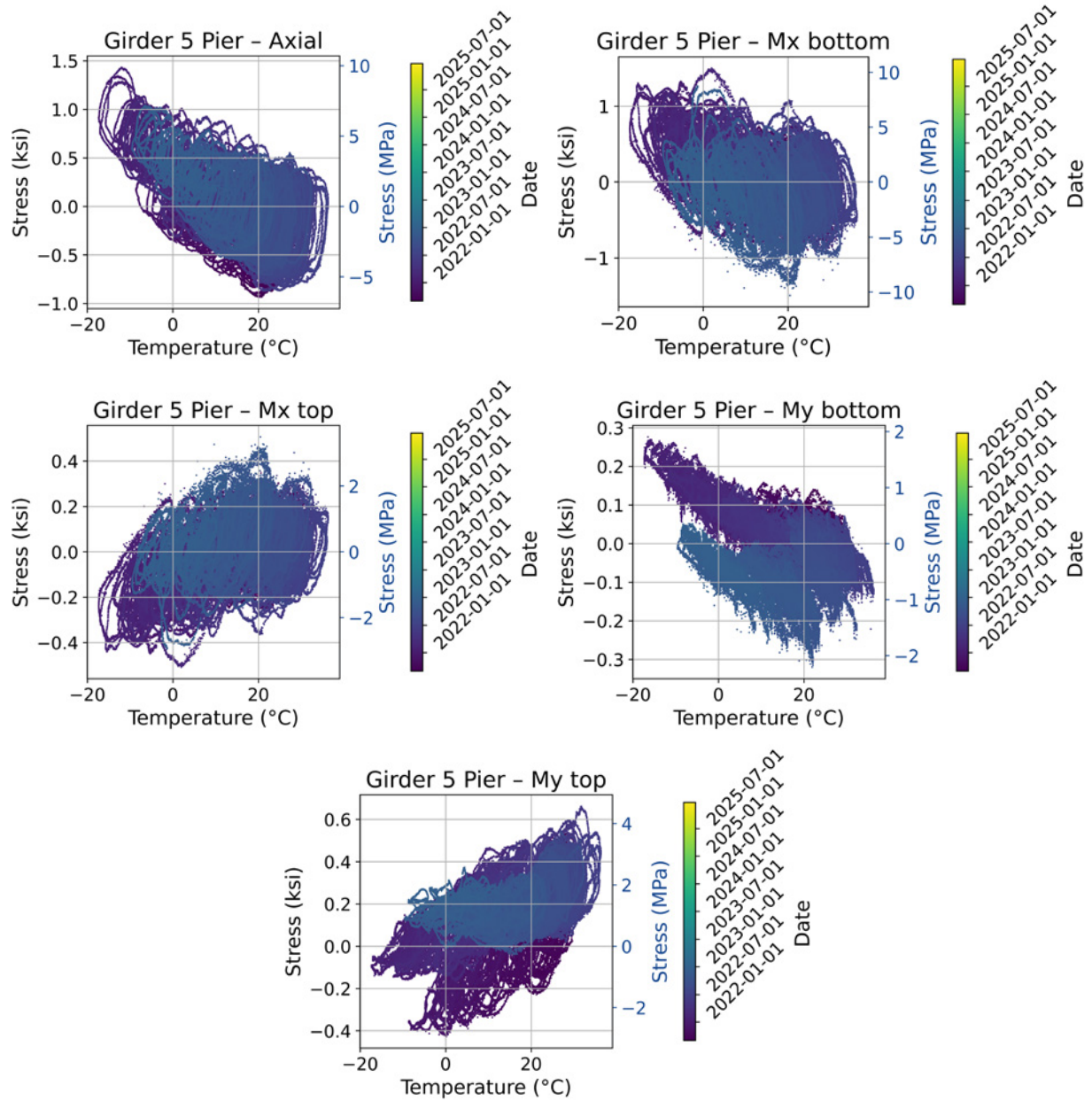


Figure 108. Graph. Temperature vs. decomposed stress data collected from Girder 5 Pier.

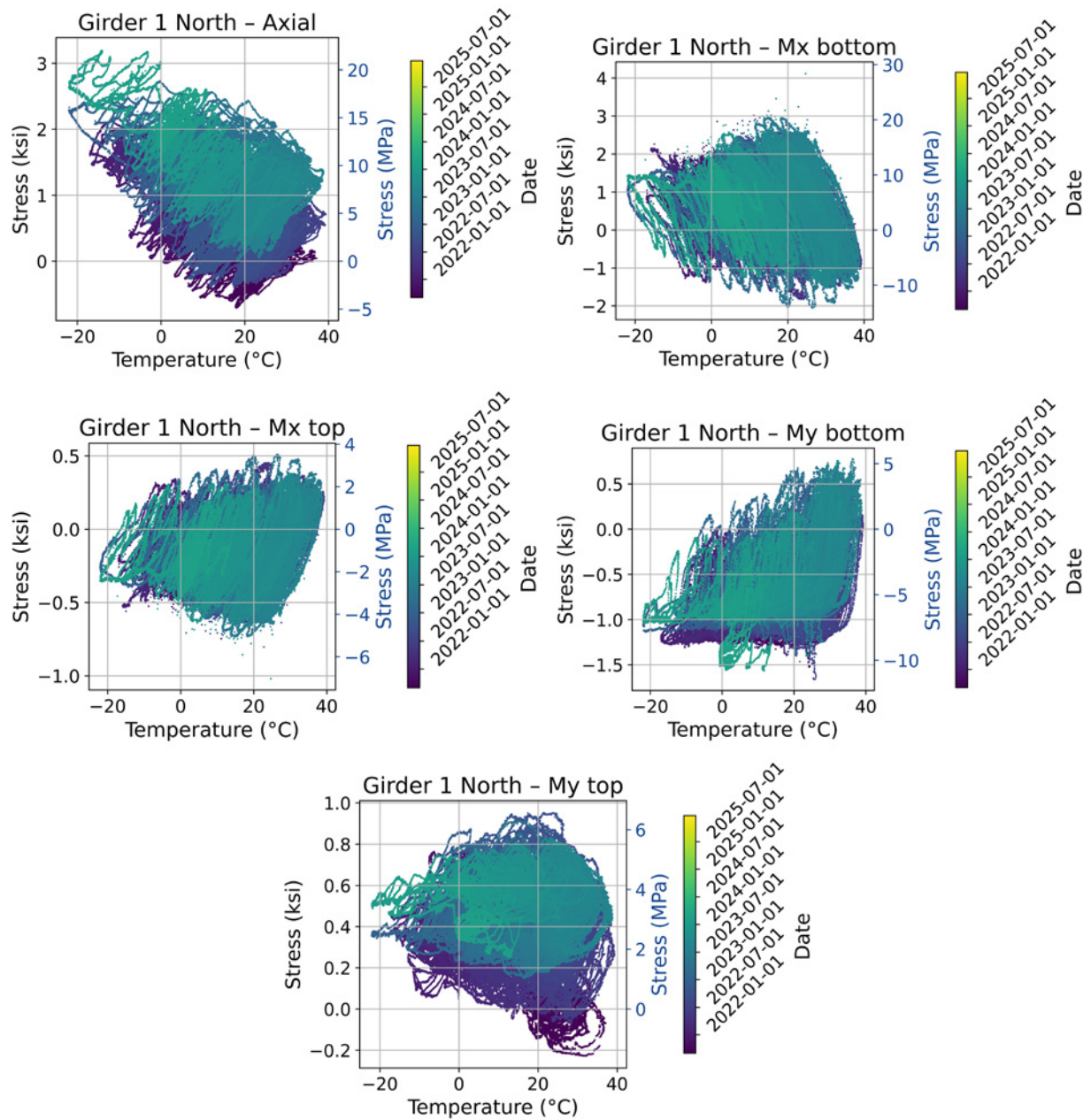


Figure 109. Graph. Temperature vs. decomposed stress data collected from Girder 1 North Span.

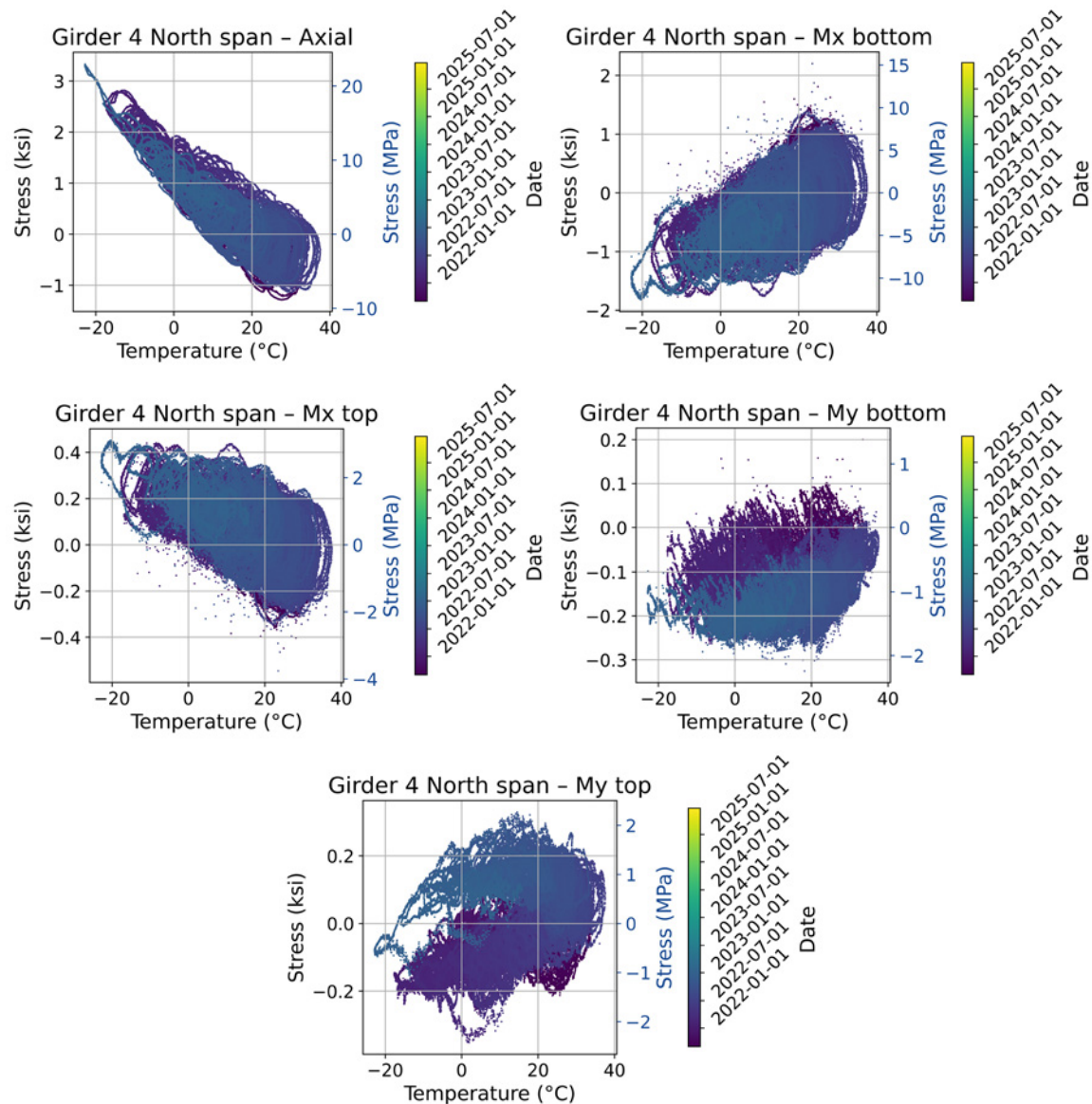


Figure 110. Graph. Temperature vs. decomposed stress data collected from Girder 4 North Span.

Cross-Frames

A total of 19 strain gauges were mounted on 6 different steel cross-frames. Three strain gauges were typically installed at each instrumented cross-frame location: at the west top portion of one diagonal angle (W), at the east top portion of the other diagonal angle (E), and at mid-length of the bottom chord angle on the concentric leg (Bc) (Figure 3). For location “CF1-1&2” (Figure 1) on the Mattis-74 bridge, an additional strain gauge was placed at mid-length of the bottom chord angle on the eccentric leg (Be in Figure 3).

Unlike the girder instrumentation discussed previously, readings exhibiting potential strain gauge malfunction were not removed from the cross-frame dataset. Consequently, some plots (and summary tables) in this section may include localized deviations or long-term drift behavior that are

not necessarily representative of the actual structural response, but are retained herein to provide a complete representation of the recorded measurements.

To provide an overview of the recorded response, Table 24 summarizes the key measures for all cross-frame gauges, including maximum, minimum, and overall range of stress values over the monitoring period.

Table 24. Summary of Stress Data from Cross-Frame Gauges—Mattis-74

Cross Frame	Member	Min. Stress MPa (ksi)	Temp. at Min. (°C)	Max. Stress MPa (ksi)	Temp. at Max. (°C)	Stress Range MPa (ksi)
CF1 1&2	B	-14.5 (-2.1)	-8.6	39.5 (5.7)	24.8	53.5 (7.8)
CF1 1&2	E	-16 (-2.3)	8.6	5.5 (0.8)	22.3	21 (3.1)
CF12 3&4	B	-5.5 (-0.8)	-2.1	20 (2.9)	17.4	25.5 (3.7)
CF12 3&4	E	-15 (-2.2)	28.3	24.5 (3.6)	-11.3	39.5 (5.8)
CF12 3&4	W	-40.5 (-5.9)	24.0	26 (3.7)	-8.0	66 (9.6)
CF12 4&5	E	-25 (-3.6)	19.0	22 (3.2)	-8.0	47 (6.8)
CF12 4&5	W	-6 (-0.9)	13.9	24 (3.5)	6.7	30 (4.4)
CF14 1&2	B	-4 (-0.6)	-3.2	23 (3.4)	22.8	27 (4)
CF14 1&2	E	-11 (-1.6)	11.7	6.5 (0.9)	23.0	18 (2.5)
CF14 1&2	W	-5.5 (-0.8)	18.4	31 (4.5)	0.9	36.5 (5.3)
CF8 3&4	B	-15.5 (-2.3)	32.8	53.5 (7.8)	-17.9	69 (10)
CF8 3&4	E	-19 (-2.7)	32.0	30.5 (4.4)	-22.8	49 (7.1)
CF8 3&4	W	-22.5 (-3.3)	24.6	11.5 (1.7)	32.5	34 (4.9)
CF8 4&5	E	-24.5 (-3.6)	27.5	40.5 (5.8)	-12.4	65 (9.4)
CF8 4&5	W	-10.5 (-1.6)	22.4	29 (4.2)	5.0	39.5 (5.8)

All data corresponding to the summary presented in Table 24 is illustrated in the response history plots below (Figure 111 through Figure 116), which show the full temporal evolution of the recorded stresses.

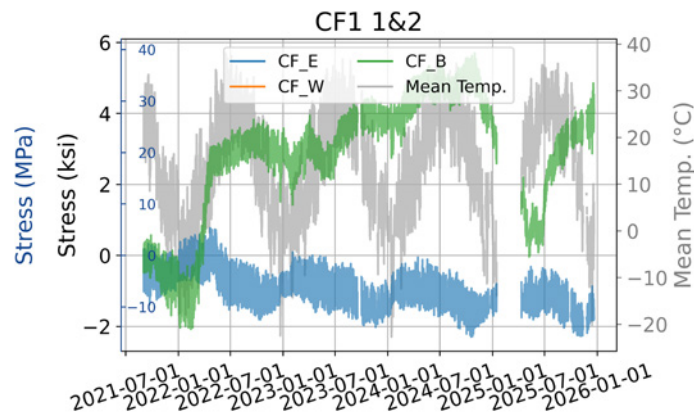


Figure 111. Graph. Stress data collected from CF1 1&2.

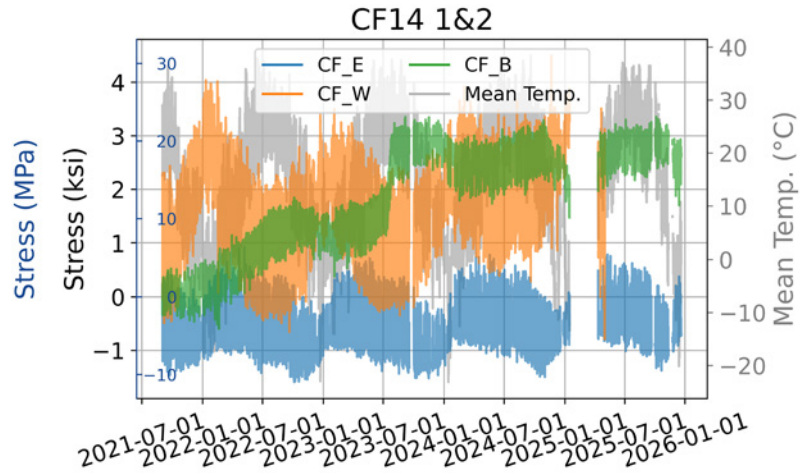


Figure 112. Graph. Stress data collected from CF14 1&2.

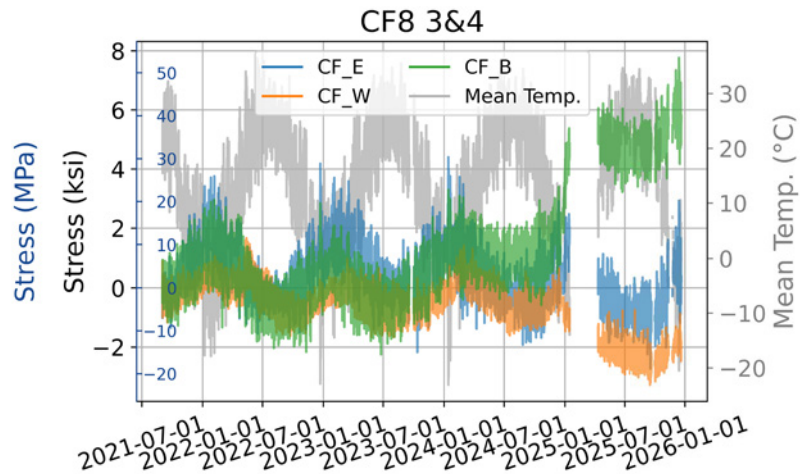


Figure 113. Graph. Stress data collected from CF8 3&4.

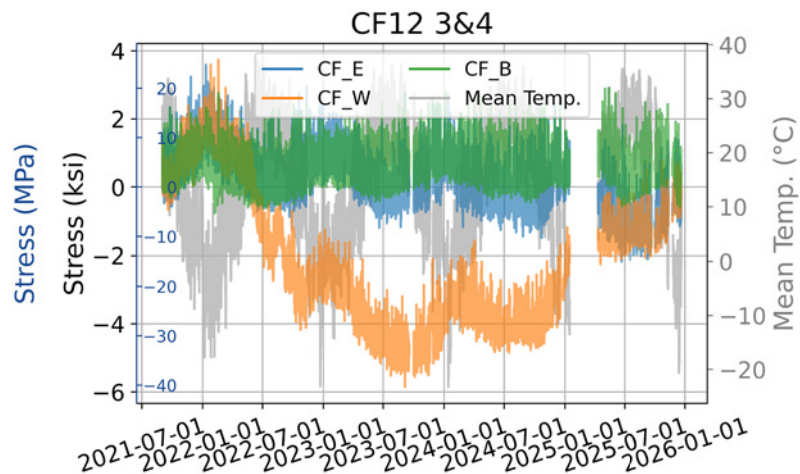


Figure 114. Graph. Stress data collected from CF12 3&4.

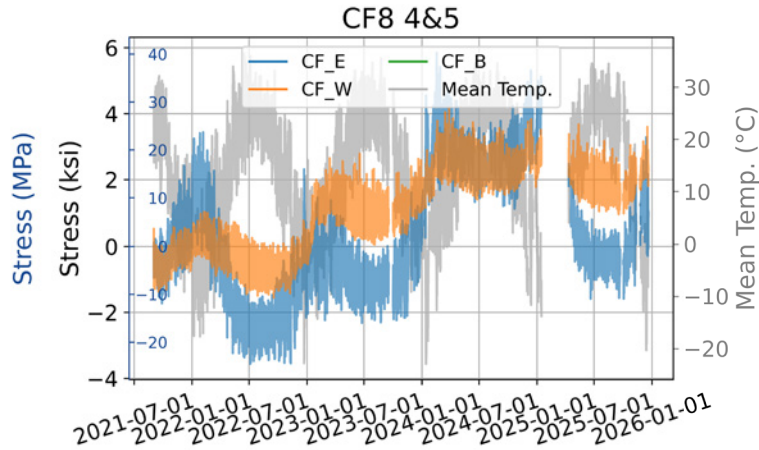


Figure 115. Graph. Stress data collected from CF8 4&5.

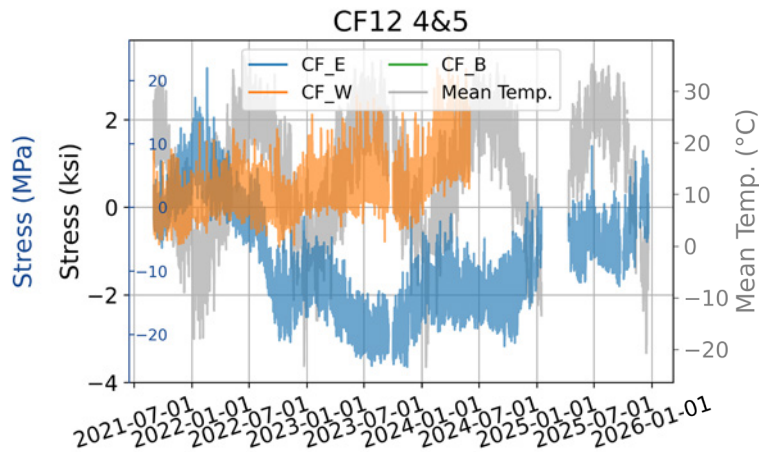


Figure 116. Graph. Stress data collected from CF12 4&5.

The same stress data presented above is presented in temperature versus decomposed stress readings in the plots below (Figure 117–Figure 122). The temperature used for these plots is the average temperature collected from thermistors attached to all gauges mounted on the cross-frame.

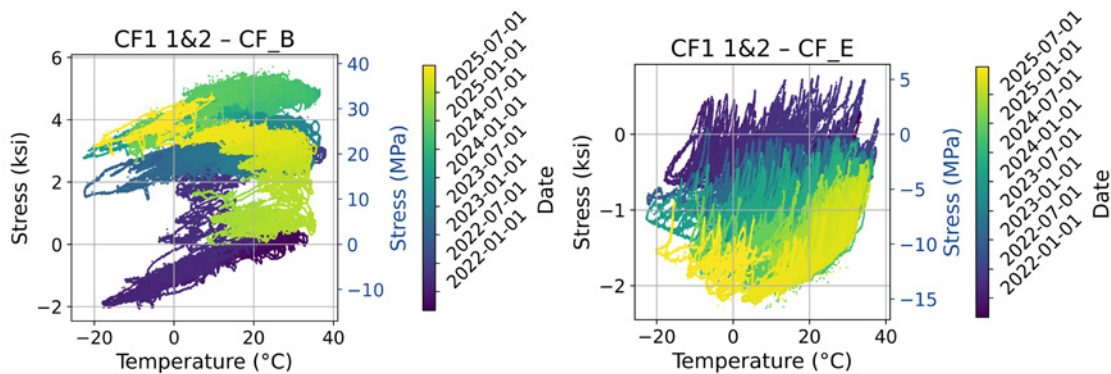


Figure 117. Graph. Temperature vs. stress data collected from CF1 1&2.

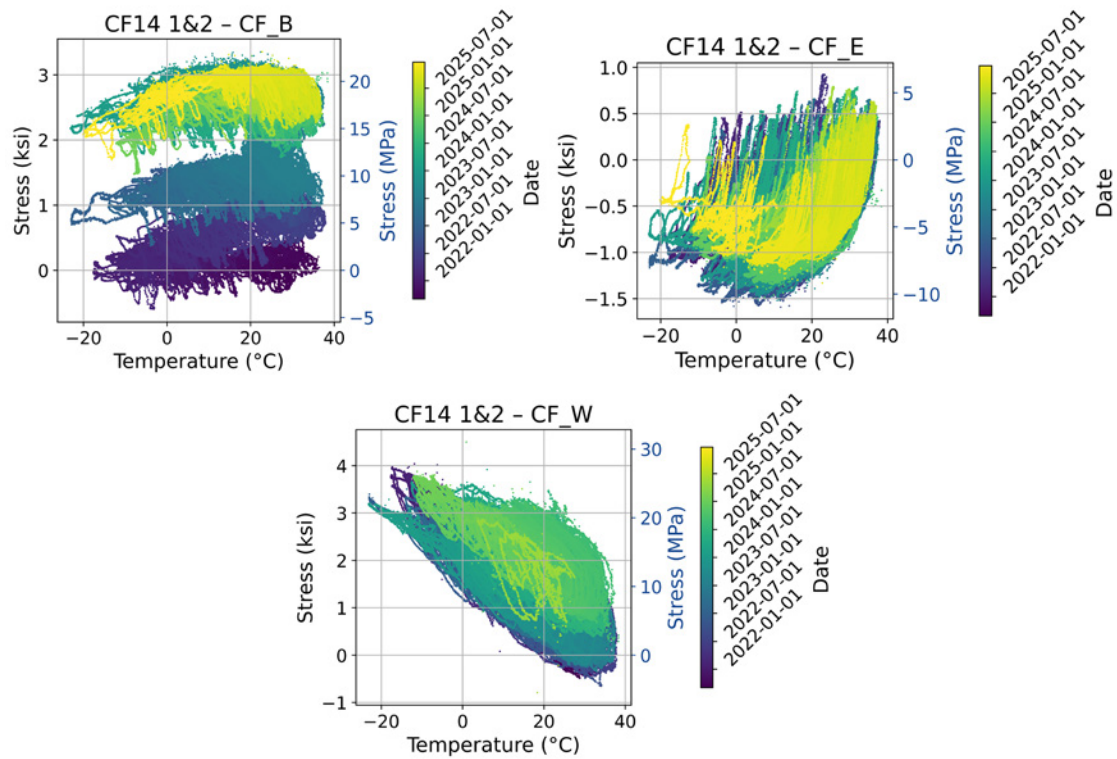


Figure 118. Graph. Temperature vs. stress data collected from CF14 1&2.

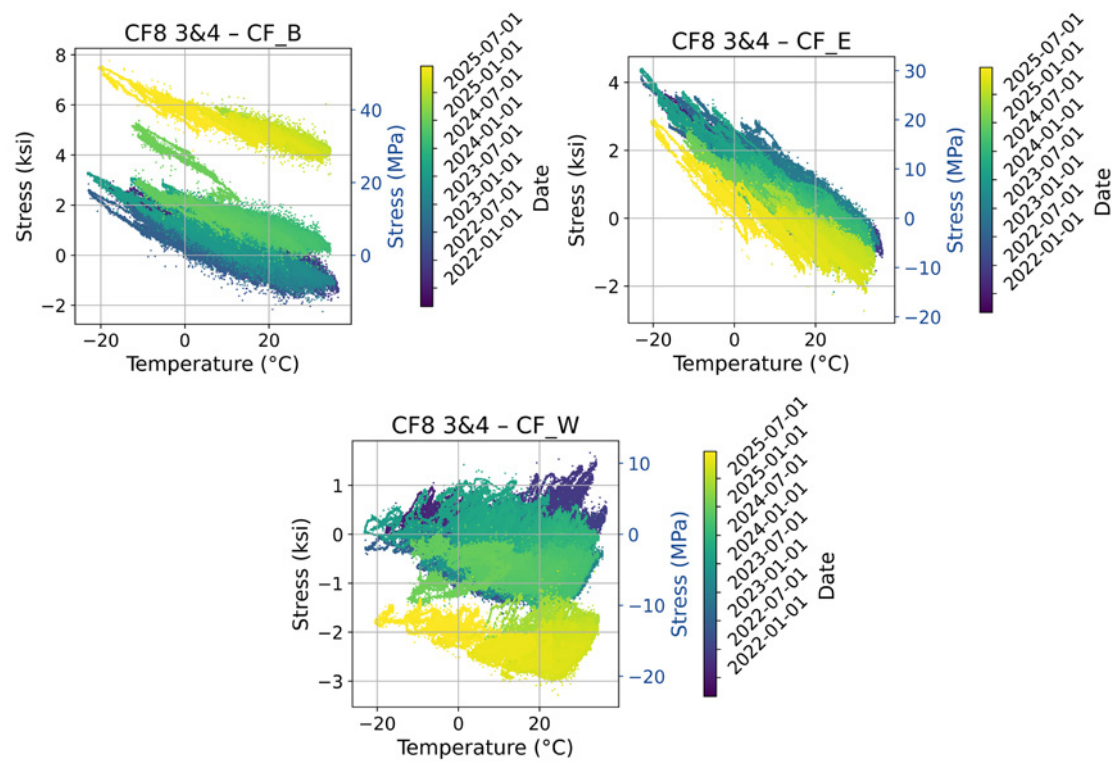


Figure 119. Graph. Temperature vs. stress data collected from CF8 3&4.

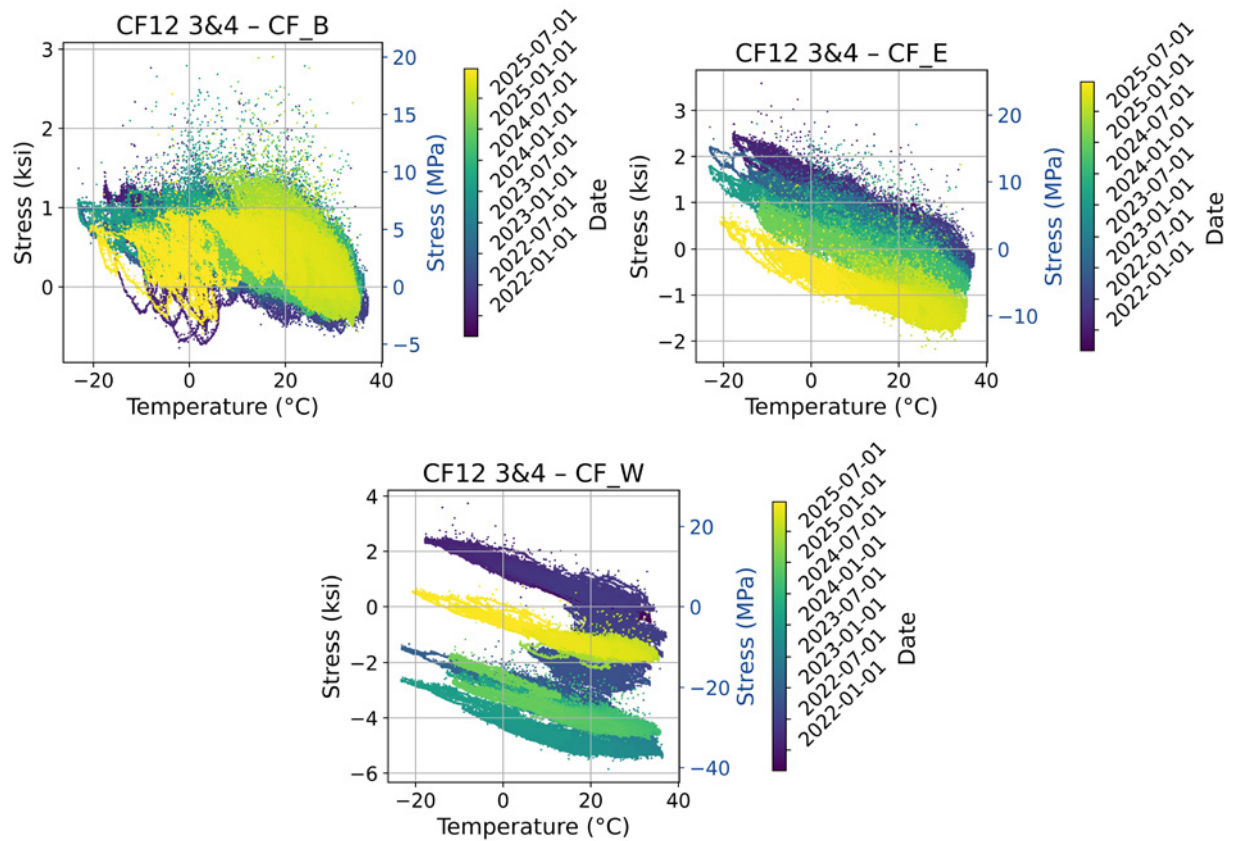


Figure 120. Graph. Temperature vs. stress data collected from CF12 3&4.

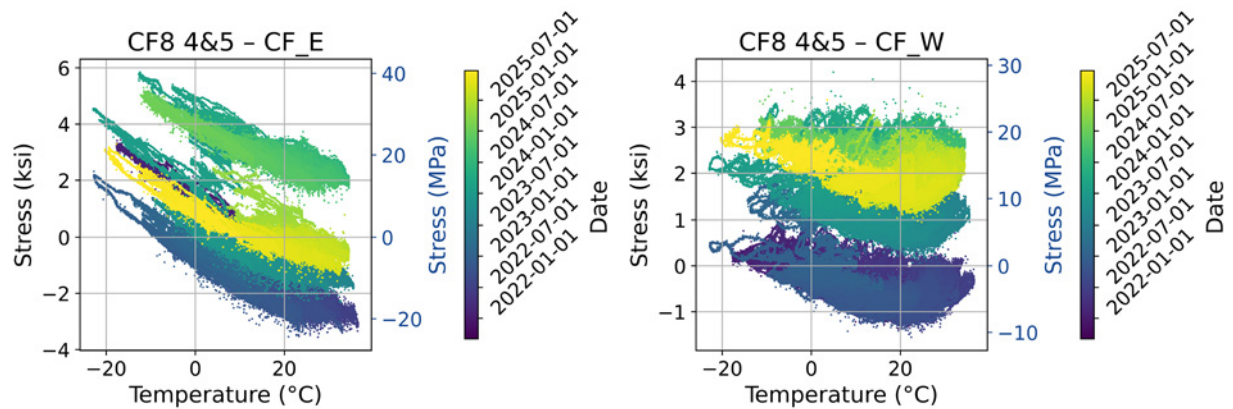


Figure 121. Graph. Temperature vs. stress data collected from CF8 4&5.

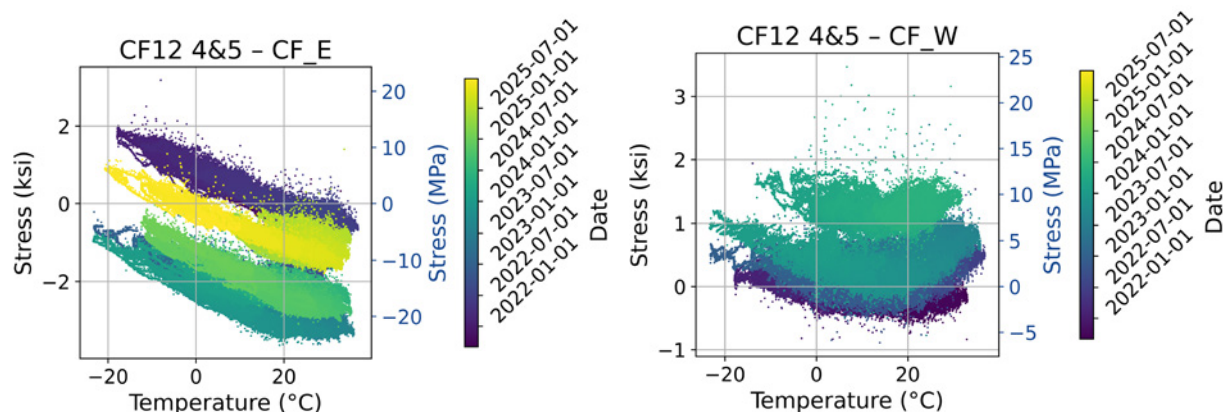


Figure 122. Graph. Temperature vs. stress data collected from CF12 4&5.

Crackmeters

A total of 8 crackmeters (denoted as “C” in Figure 1) were installed on Mattis-74 bridge. These were installed to measure relative displacement between the bottom flange of both exterior girders and the abutment backwall, both in the longitudinal and perpendicular directions (parallel and perpendicular to girder). The installation process and exact locations are described in detail in Volume I (Zhou et al. 2026b).

To provide an overview of the recorded response, Table 25 summarizes the key measures for all crackmeters, including maximum, minimum, and overall range of displacement values over the monitoring period.

Table 25. Summary of Crackmeter Displacement Data—Mattis-74

Orientation	Location	Corner Type	Min. Displ. cm (in.)	Temp. at Min. (°C)	Max. Displ. cm (in.)	Temp. at Max. (°C)	Displ. Range cm (in.)
Par.	NE	obtuse	−0.93 (−0.37)	33.8	2.22 (0.87)	−21.3	3.15 (1.24)
Par.	NW	acute	−1.13 (−0.45)	37.9	2.92 (1.15)	−22.2	4.05 (1.60)
Par.	SE	acute	−1.38 (−0.54)	35.0	1.70 (0.67)	−21.2	3.09 (1.21)
Par.	SW	obtuse	−0.47 (−0.19)	40.0	3.69 (1.45)	−20.5	4.16 (1.64)
Perp.	NE	obtuse	−0.18 (−0.07)	33.2	0.18 (0.07)	−16.3	0.36 (0.14)
Perp.	NW	acute	−0.38 (−0.15)	33.3	0.44 (0.17)	−16.6	0.83 (0.33)
Perp.	SE	acute	−0.16 (−0.06)	30.0	0.24 (0.09)	30.2	0.39 (0.15)
Perp.	SW	obtuse	−0.07 (−0.03)	34.4	0.22 (0.09)	−22.9	0.29 (0.11)

All data corresponding to the summary presented in Table 25 is illustrated in the response history plots below (Figures 123 through 126), which show the full temporal evolution of the recorded displacements.

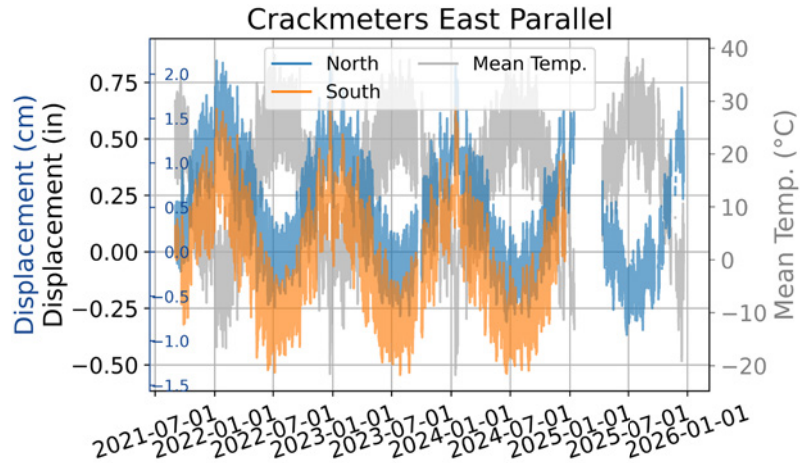


Figure 123. Graph. Displacement data collected from east crackmeters (parallel).

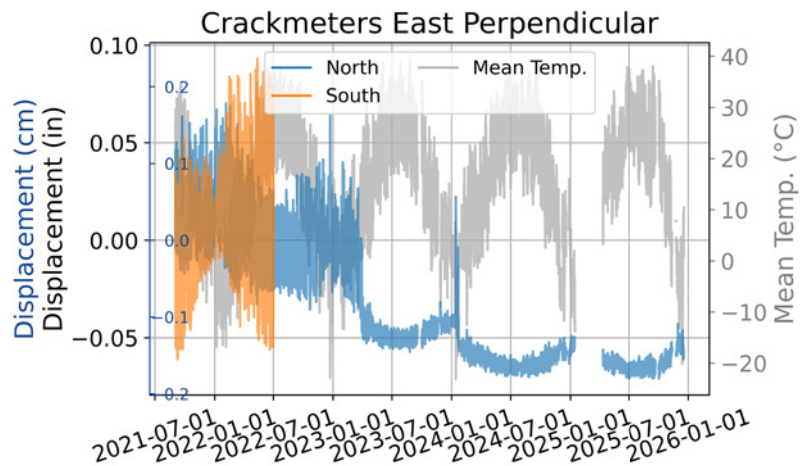


Figure 124. Graph. Displacement data collected from east crackmeters (perpendicular).

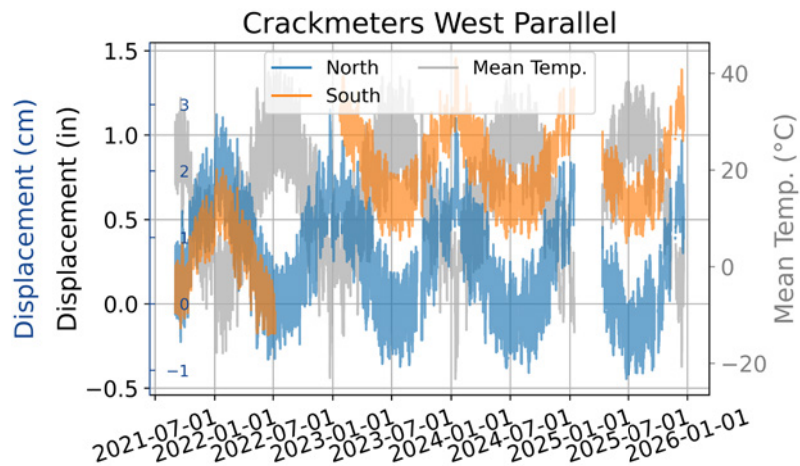


Figure 125. Graph. Displacement data collected from west crackmeters (parallel).

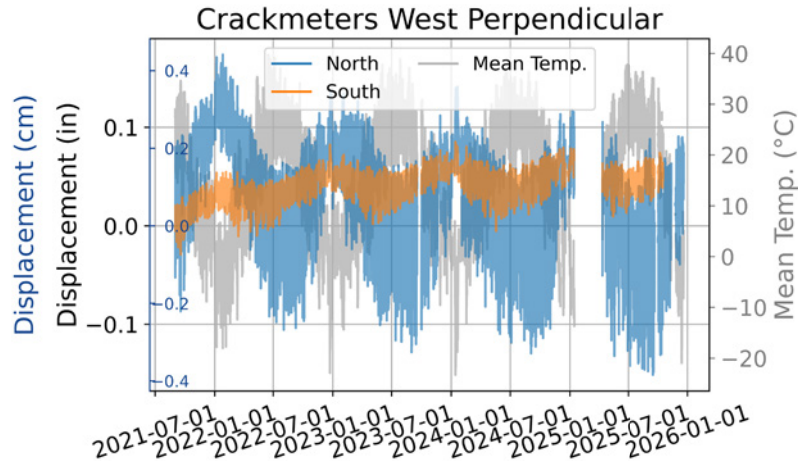


Figure 126. Graph. Displacement data collected from west crackmeters (perpendicular).

The same displacement data presented above is presented in temperature versus displacement readings in Figure 127 through Figure 130. The temperature used for these plots is the average temperature collected from thermistors attached to all crackmeters on each plot.

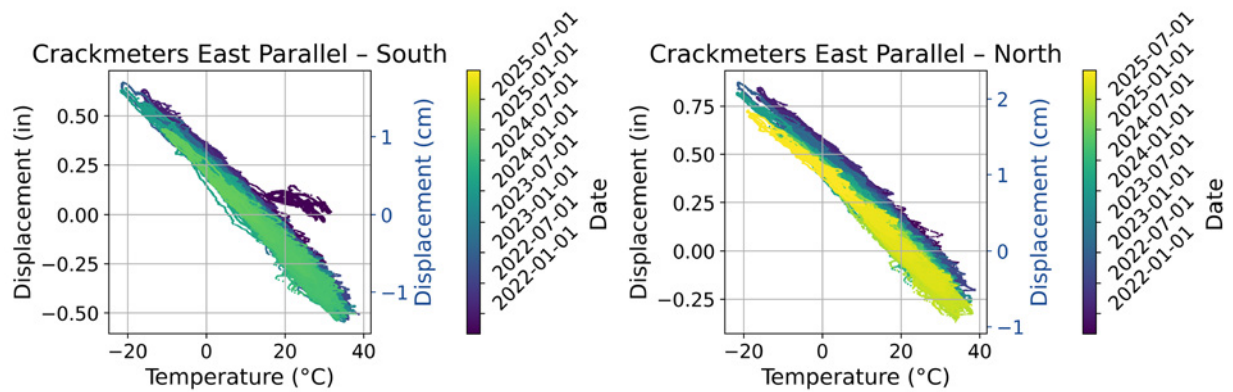


Figure 127. Graph. Temperature vs. displacement data collected from east crackmeters (parallel).

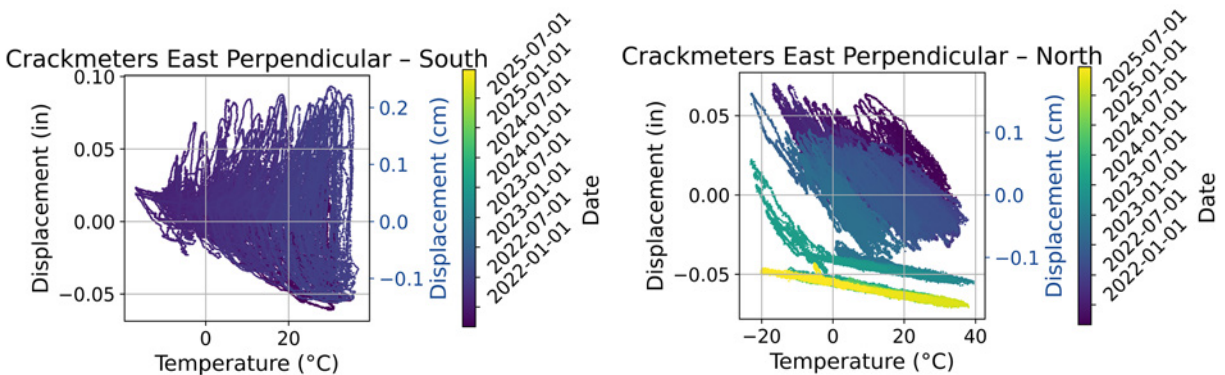


Figure 128. Graph. Temperature vs. displacement data collected from east crackmeters (perpendicular).

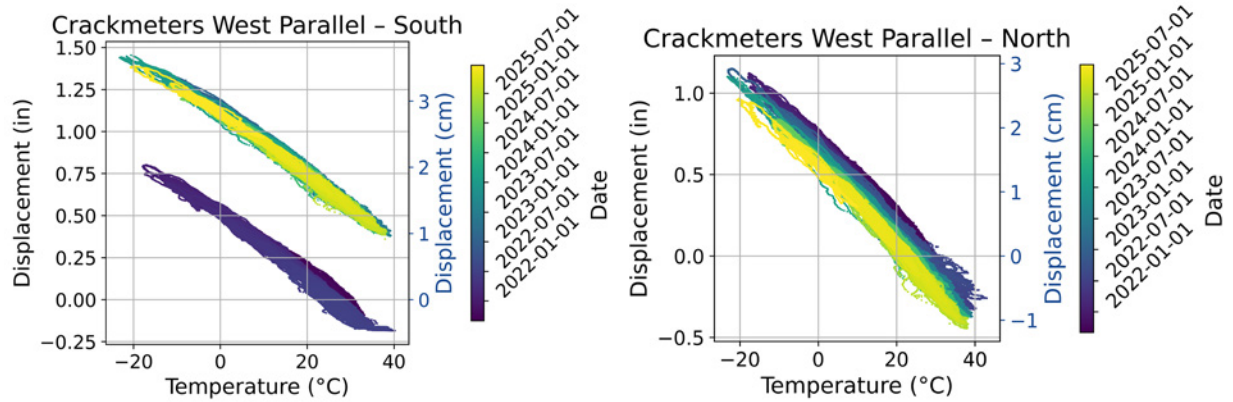


Figure 129. Graph. Temperature vs. displacement data collected from west crackmeters (parallel).

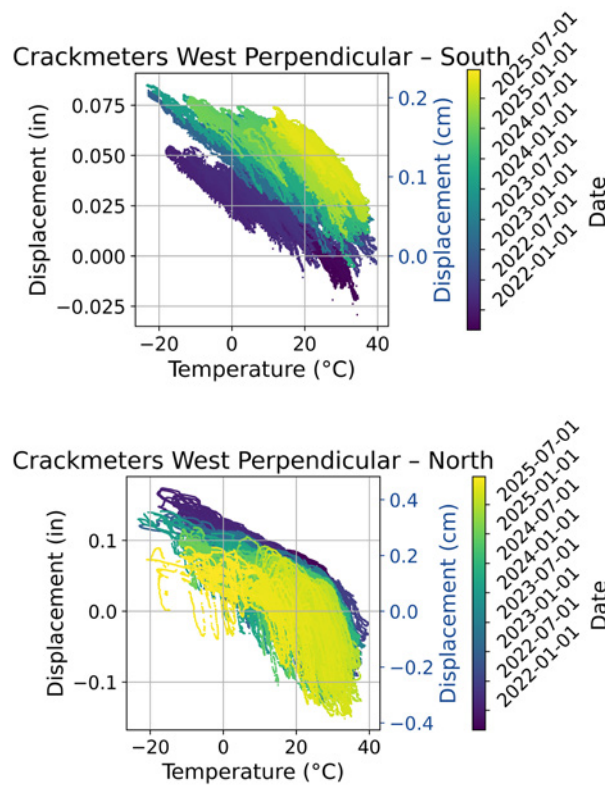


Figure 130. Graph. Temperature vs. displacement data collected from west crackmeters (perpendicular).

Tiltmeters

A total of 2 tiltmeters (denoted as “T” in Figure 1) were installed on Mattis-74 bridge. These were installed to measure rotation at the girder ends, and were mounted at mid-height on the end bearing stiffeners of girder 5 (G5 in Figure 1).

To provide an overview of the recorded response, Table 26 summarizes the key measures for both tiltmeters, including maximum, minimum, and overall range of rotation values over the monitoring period.

Table 26. Summary of Rotation Measurements—Mattis-74

Location	Min. Rotation (deg)	Temp. at Min. (°C)	Max. Rotation (deg)	Temp. at Max. (°C)	Rotation Range (deg)
North	-0.05	30.4	0.14	-22.0	0.19
South	-0.05	30.2	0.11	-22.0	0.17

All data corresponding to the summary presented in Table 26 is illustrated in the response history plot below (Figure 131), which show the full temporal evolution of the recorded rotations.

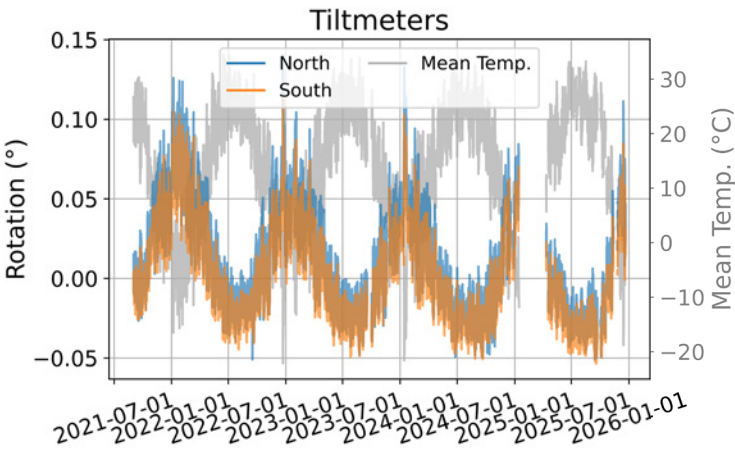


Figure 131. Graph. Rotation data collected from tiltmeters.

The same rotation data presented above, is presented in temperature versus rotation form in Figure 132. The temperature used for these plots is the average temperature collected from thermistors attached to all tiltmeters on each plot.

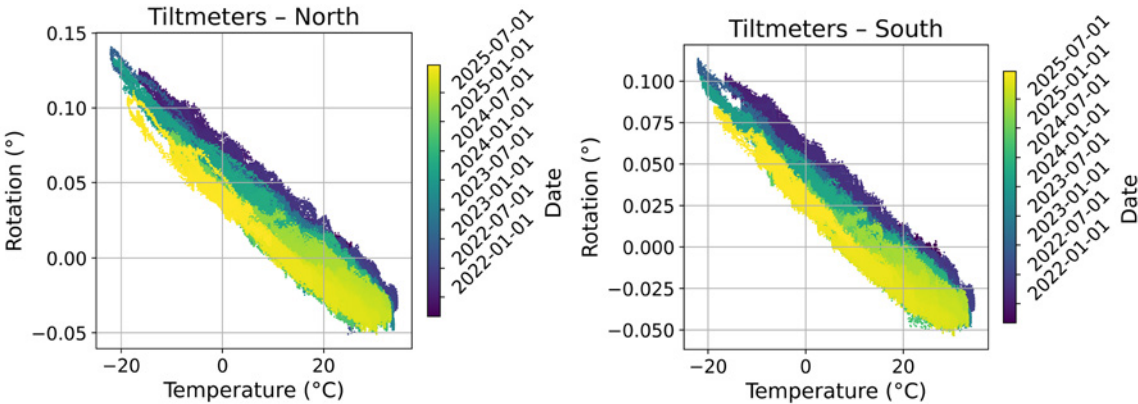


Figure 132. Graph. Temperature vs. rotation data collected from tiltmeters.

DATA COLLECTED FROM MATTIS-57 (IAB)

Stress Data from Girder Cross-Sections

A total of 14 girder cross-sections were instrumented in Mattis-57 bridge, 3 of which were heavily instrumented, as described in Chapter 1. The total number of strain gauges mounted on girder cross-sections sums up to 67 (64 mounted on steel I-girders and 3 embedded in concrete deck).

To provide an overview of the recorded response, Table 27 summarizes the key measures for all gauges, including maximum, minimum, and overall range of stress values over the monitoring period.

Table 27. Summary of Stress Data from Girder Cross-Section Gauges—Mattis-57

Cross Section	Gauge	Min. Stress MPa (ksi)	Temp. at Min. (°C)	Max. Stress MPa (ksi)	Temp. at Max. (°C)	Stress Range MPa (ksi)
G5 Pier N.	BEt	−7 (−1)	23.5	12.5 (1.8)	−16.8	19.5 (2.8)
G5 Pier N.	TE	−13.5 (−2)	31.4	24 (3.5)	−21.9	37.5 (5.5)
G5 Pier S.	BWb	−7.5 (−1.1)	16.2	18 (2.6)	−17.3	25.5 (3.7)
G5 Pier S.	TE	−11.5 (−1.7)	19.4	6.5 (0.9)	4.7	18 (2.6)
G5 S. Abut.	BEb	−30 (−4.3)	34.9	43 (6.2)	−22.9	73 (10.6)
G5 S. Abut.	TE	−11.5 (−1.7)	9.5	10 (1.4)	2.3	21.5 (3.1)
G5 S. Abut.	TW	−10.5 (−1.6)	34.0	29.5 (4.3)	−11.6	40.5 (5.9)
G5 S. Span	BEb	−9 (−1.3)	29.8	24 (3.5)	−17.6	33 (4.8)
G5 S. Span	BWb	−8.5 (−1.2)	30.6	27 (3.9)	−1.7	35.5 (5.2)
G5 S. Span	TE	−15 (−2.2)	28.0	23.5 (3.4)	−22.4	38.5 (5.6)
G5 S. Span	TW	−9 (−1.3)	29.4	32 (4.6)	−12.0	41 (6)
G6 Pier N.	BWt	−8.5 (−1.3)	22.6	24.5 (3.6)	−22.5	33.5 (4.8)
G6 Pier N.	TE	−11 (−1.6)	31.7	22.5 (3.3)	−22.6	33.5 (4.8)
G6 Pier S.	BEb	−9 (−1.3)	23.0	8 (1.2)	2.6	17 (2.5)
G6 Pier S.	BEt	−6.5 (−1)	23.0	14.5 (2.1)	−18.2	21 (3.1)
G6 Pier S.	BWb	−8.5 (−1.2)	23.0	7 (1)	2.4	16 (2.3)
G6 Pier S.	BWt	−8 (−1.2)	23.0	18 (2.6)	−0.2	26 (3.7)
G6 Pier S.	TE	−10.5 (−1.5)	32.1	9.5 (1.4)	−16.5	20 (2.9)
G6 Pier S.	TW	−10 (−1.5)	31.6	19.5 (2.8)	−15.0	29.5 (4.3)
G6 S. Abut.	BEb	−17 (−2.5)	21.6	34 (4.9)	−17.3	51 (7.4)
G6 S. Abut.	BEt	−23.5 (−3.4)	31.4	37.5 (5.4)	−23.0	61 (8.8)
G6 S. Abut.	BWb	−27.5 (−4)	31.4	31 (4.5)	−22.2	58.5 (8.5)
G6 S. Abut.	BWt	−14.5 (−2.1)	29.9	29 (4.2)	−7.4	43.5 (6.3)
G6 S. Abut.	TE	−13 (−1.9)	32.8	22.5 (3.3)	−22.8	36 (5.2)
G6 S. Abut.	TW	−12.5 (−1.8)	30.2	30 (4.3)	−19.2	42 (6.1)
G6 S. Span	BEb	−9 (−1.3)	30.0	36.5 (5.3)	−17.4	45.5 (6.6)
G6 S. Span	BEt	−11 (−1.6)	35.0	41.5 (6)	−22.3	52.5 (7.6)
G6 S. Span	BWb	−11 (−1.6)	35.4	31 (4.5)	−17.4	42 (6.1)
G6 S. Span	BWt	−12.5 (−1.8)	34.5	30.5 (4.4)	−17.4	43 (6.2)

Cross Section	Gauge	Min. Stress MPa (ksi)	Temp. at Min. (°C)	Max. Stress MPa (ksi)	Temp. at Max. (°C)	Stress Range MPa (ksi)
G6 S. Span	TE	-13.5 (-1.9)	31.4	25 (3.7)	-22.1	38.5 (5.6)
G6 S. Span	TW	-12 (-1.7)	31.7	27 (3.9)	-22.7	39 (5.6)
G7 S. Abut.	BEb	-24 (-3.5)	31.2	33.5 (4.9)	-17.5	57.5 (8.4)
G7 S. Abut.	BWb	-19 (-2.7)	31.2	25.5 (3.7)	-17.5	44.5 (6.4)
G7 S. Abut.	TE	-15 (-2.2)	27.3	16 (2.3)	-10.5	30.5 (4.4)
G7 S. Abut.	TW	-9 (-1.3)	22.9	19 (2.8)	-12.2	28 (4.1)
G8 Pier S.	BWb	-13 (-1.9)	30.9	15.5 (2.3)	3.1	29 (4.2)
G8 Pier S.	TE	-7.5 (-1.1)	26.8	13.5 (2)	-21.4	21 (3.1)
G8 Pier S.	TW	-9.5 (-1.4)	33.6	12 (1.7)	-10.5	21.5 (3.1)
G8 N. Abut.	BEb	-54.5 (-7.9)	38.5	52.5 (7.6)	-21.6	107 (15.5)
G8 N. Abut.	BWb	-30 (-4.3)	40.8	50.5 (7.3)	-21.6	80 (11.6)
G8 N. Abut.	TE	-11 (-1.6)	37.4	19 (2.7)	-21.3	30 (4.3)
G8 N. Abut.	TW	-9 (-1.3)	36.6	14.5 (2.1)	-11.8	24 (3.4)
G8 S. Abut.	BEb	-32.5 (-4.7)	33.3	38.5 (5.6)	-22.8	70.5 (10.2)
G8 S. Abut.	BWb	-20.5 (-3)	31.1	33.5 (4.9)	-17.4	54.5 (7.9)
G8 S. Abut.	TE	-8 (-1.1)	15.2	18 (2.6)	7.2	26 (3.7)
G8 S. Abut.	TW	-7 (-1)	16.5	16 (2.3)	-22.9	23.5 (3.4)
G8 S. Span	BEb	-48 (-7)	31.1	30.5 (4.4)	-16.6	78.5 (11.4)
G8 S. Span	BWb	-27.5 (-4)	32.2	37.5 (5.4)	-4.5	65 (9.4)
G8 S. Span	TE	-8.5 (-1.2)	25.2	25 (3.6)	-21.8	33 (4.8)
G8 S. Span	TW	-7 (-1)	23.8	25 (3.7)	-22.8	32 (4.7)

All data corresponding to the summary presented in Table 27 is illustrated in the response history plots below (Figure 133 through Figure 145), which show the full temporal evolution of the recorded stresses.

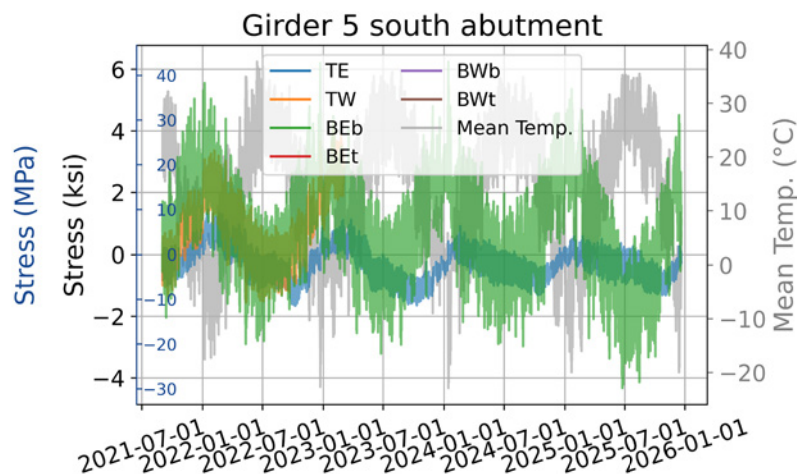


Figure 133. Graph. Stress data collected from Girder 5 South Abutment.

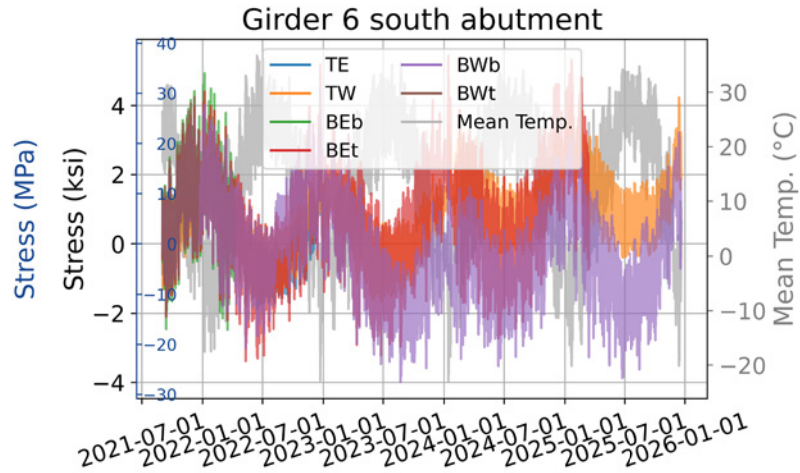


Figure 134. Graph. Stress data collected from Girder 6 South Abutment.

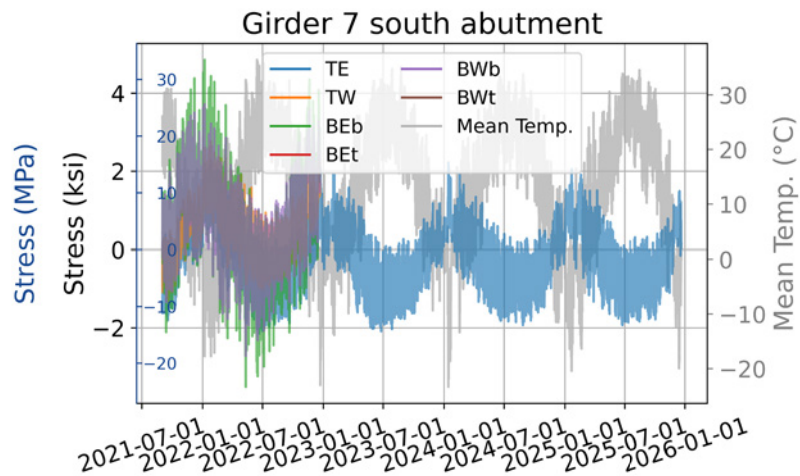


Figure 135. Graph. Stress data collected from Girder 7 South Abutment.

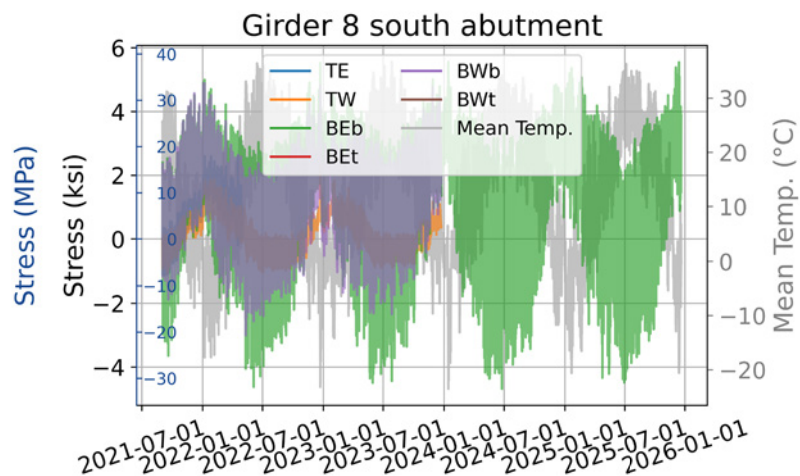


Figure 136. Graph. Stress data collected from Girder 8 South Abutment.

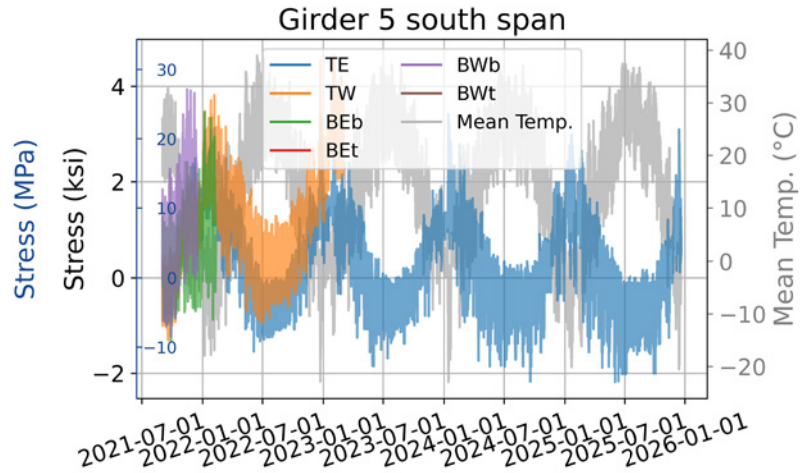


Figure 137. Graph. Stress data collected from Girder 5 South Span.

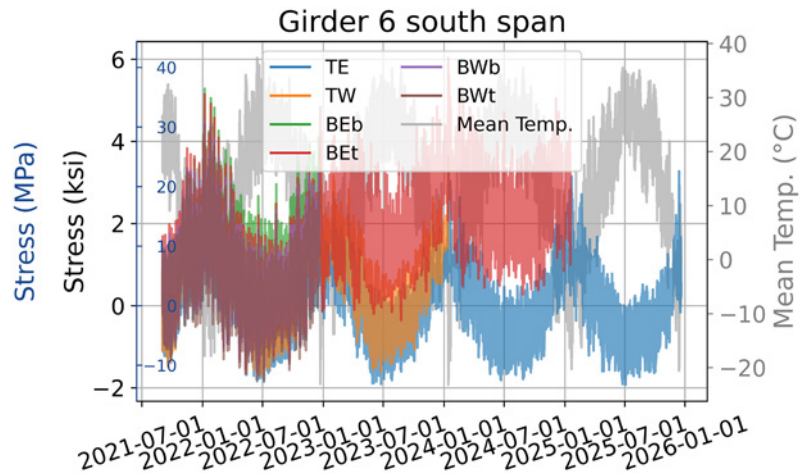


Figure 138. Graph. Stress data collected from Girder 6 South Span.

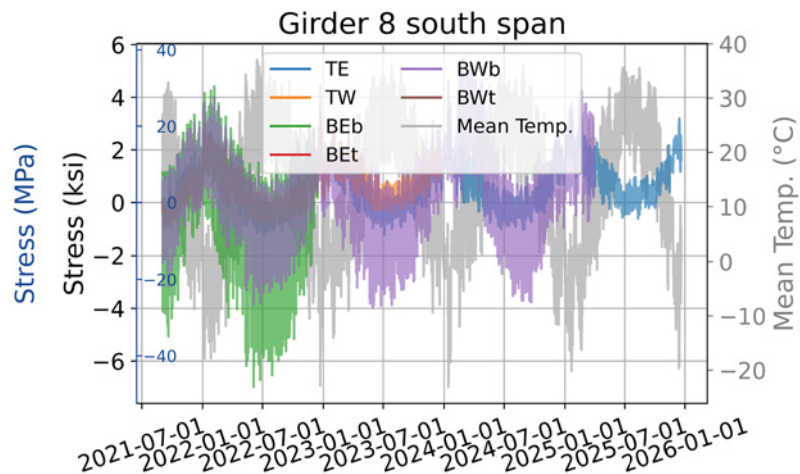


Figure 139. Graph. Stress data collected from Girder 8 South Span.

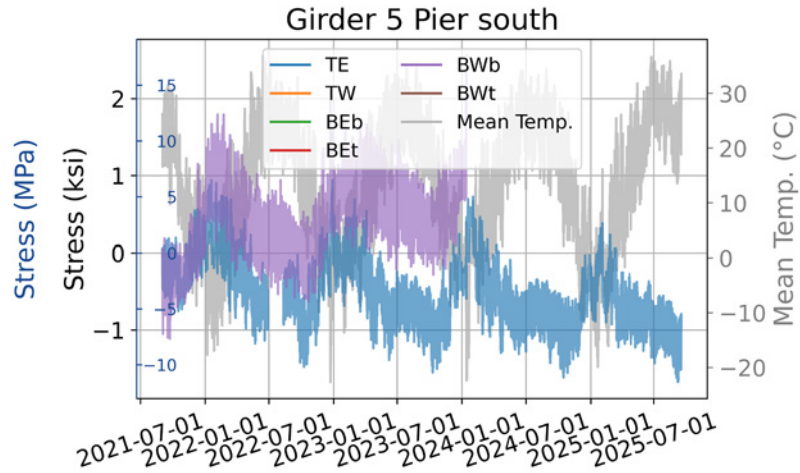


Figure 140. Graph. Stress data collected from Girder 5 Pier South.

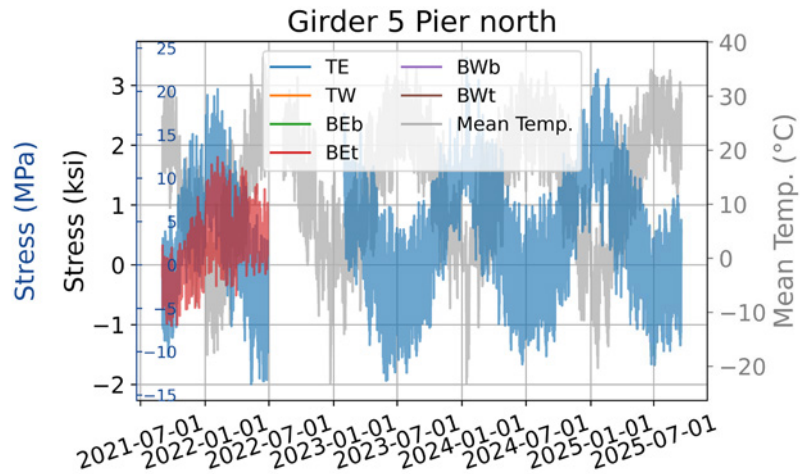


Figure 141. Graph. Stress data collected from Girder 5 Pier North.

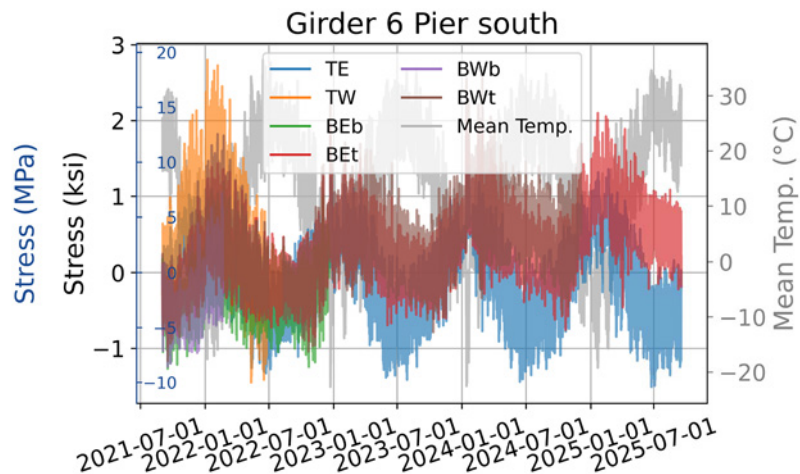


Figure 142. Graph. Stress data collected from Girder 6 Pier South.

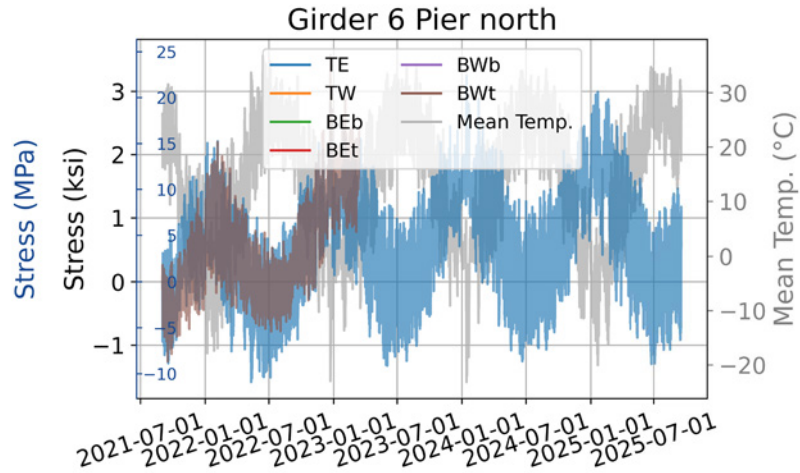


Figure 143. Graph. Stress data collected from Girder 6 Pier North.

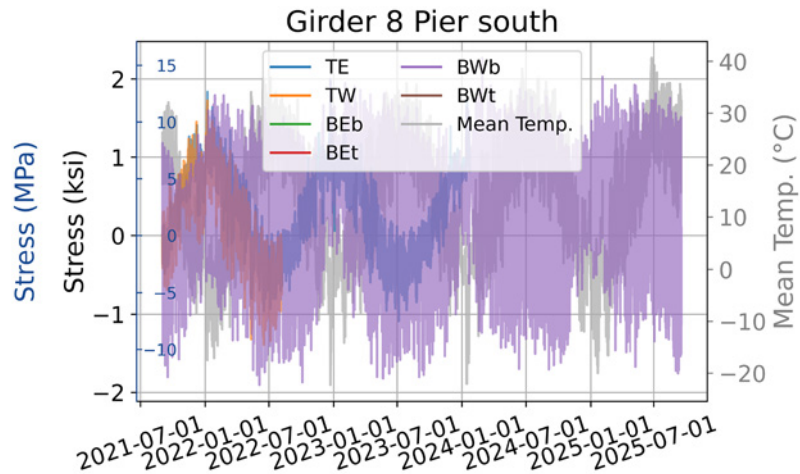


Figure 144. Graph. Stress data collected from Girder 8 Pier.

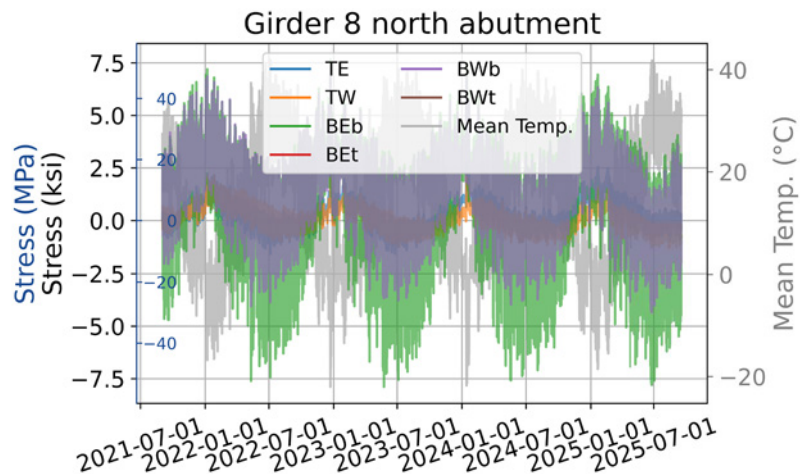


Figure 145. Graph. Stress data collected from Girder 8 North Abutment.

The same data presented above, is presented in temperature versus stress form in Figure 146 through Figure 158. The temperature used for these plots is the average temperature collected from thermistors attached to all gauges mounted on the cross-section.

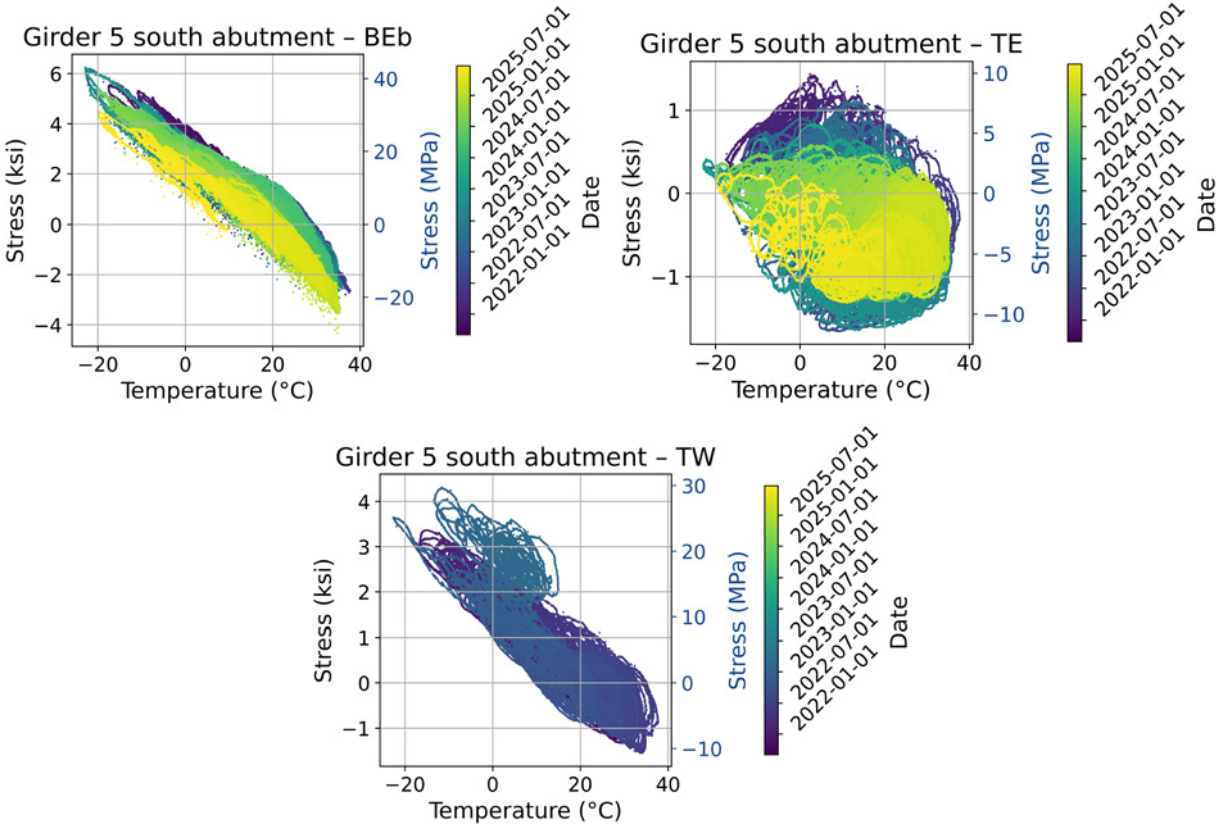


Figure 146. Graph. Temperature vs. stress data collected from Girder 5 South Abutment.

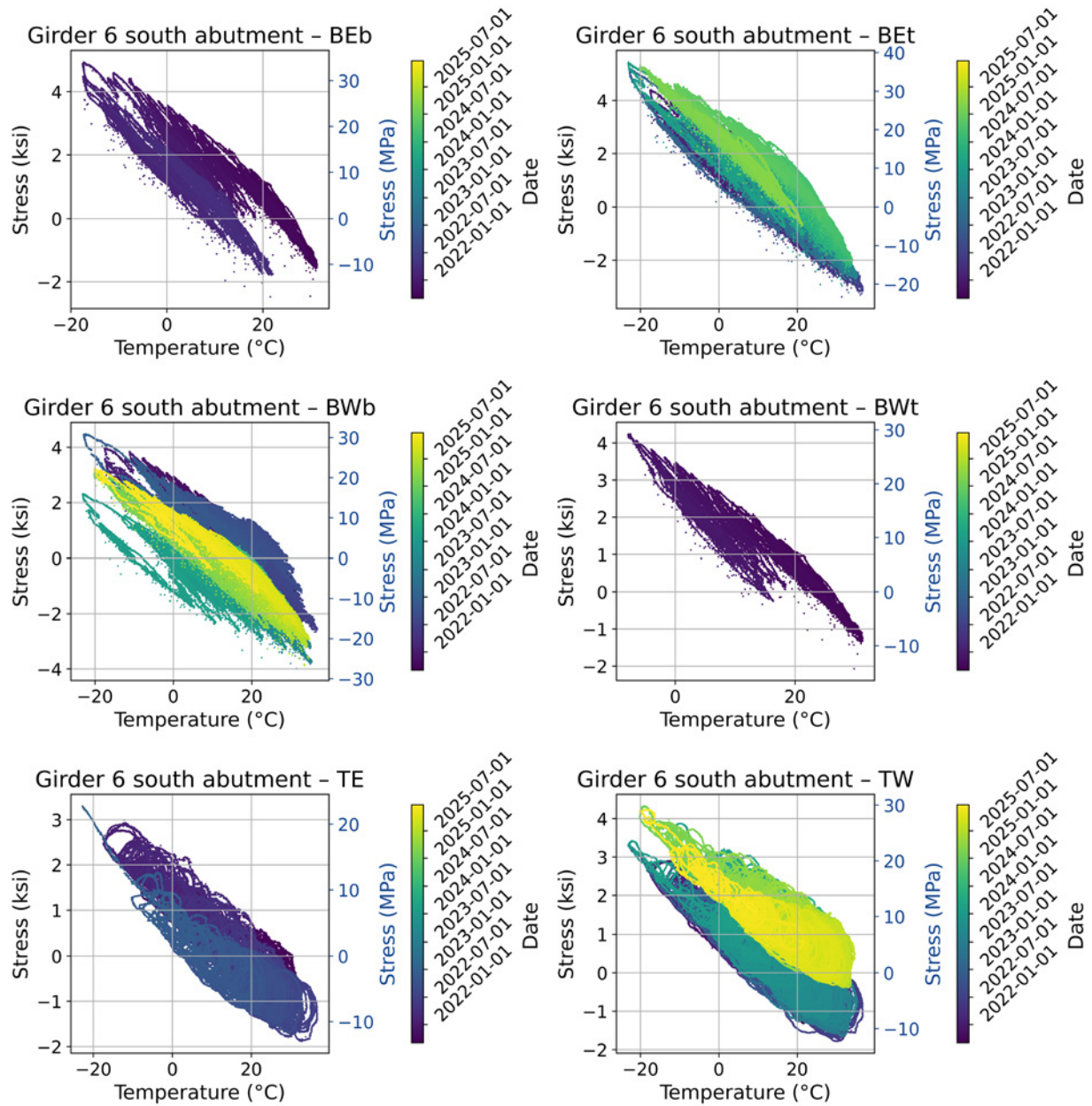


Figure 147. Graph. Temperature vs. stress data collected from Girder 6 South Abutment.

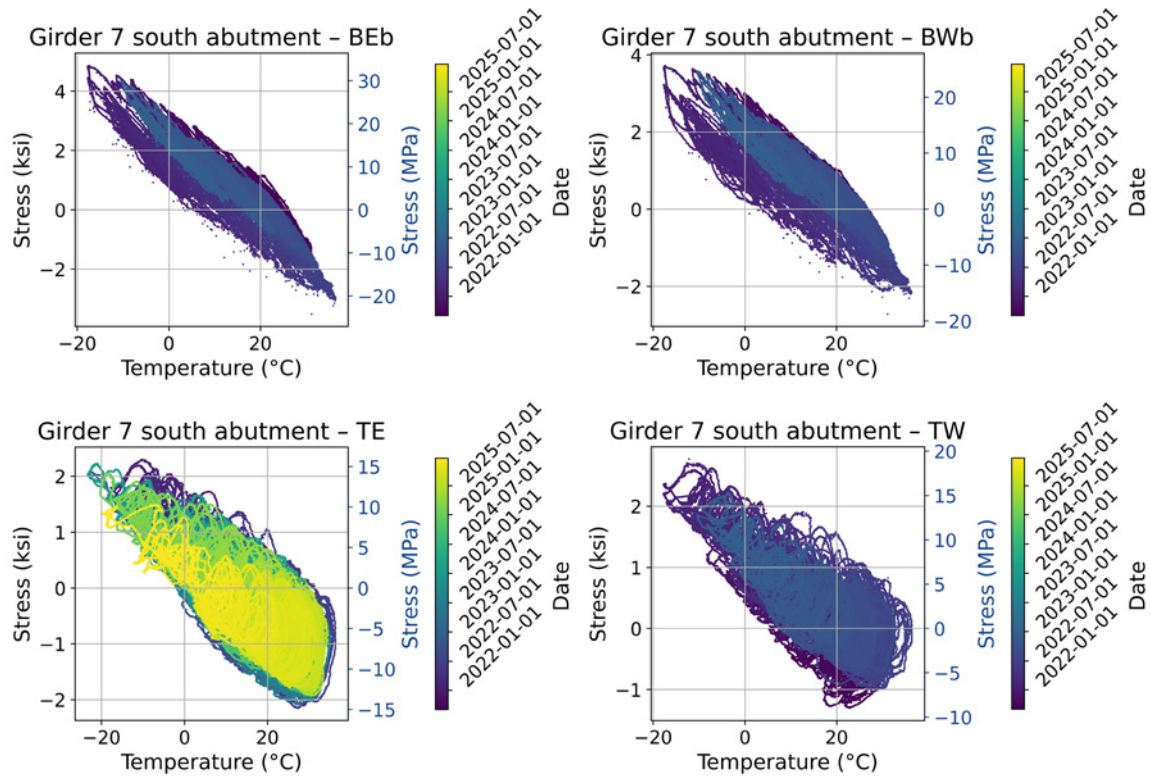


Figure 148. Graph. Temperature vs. stress data collected from Girder 7 South Abutment.

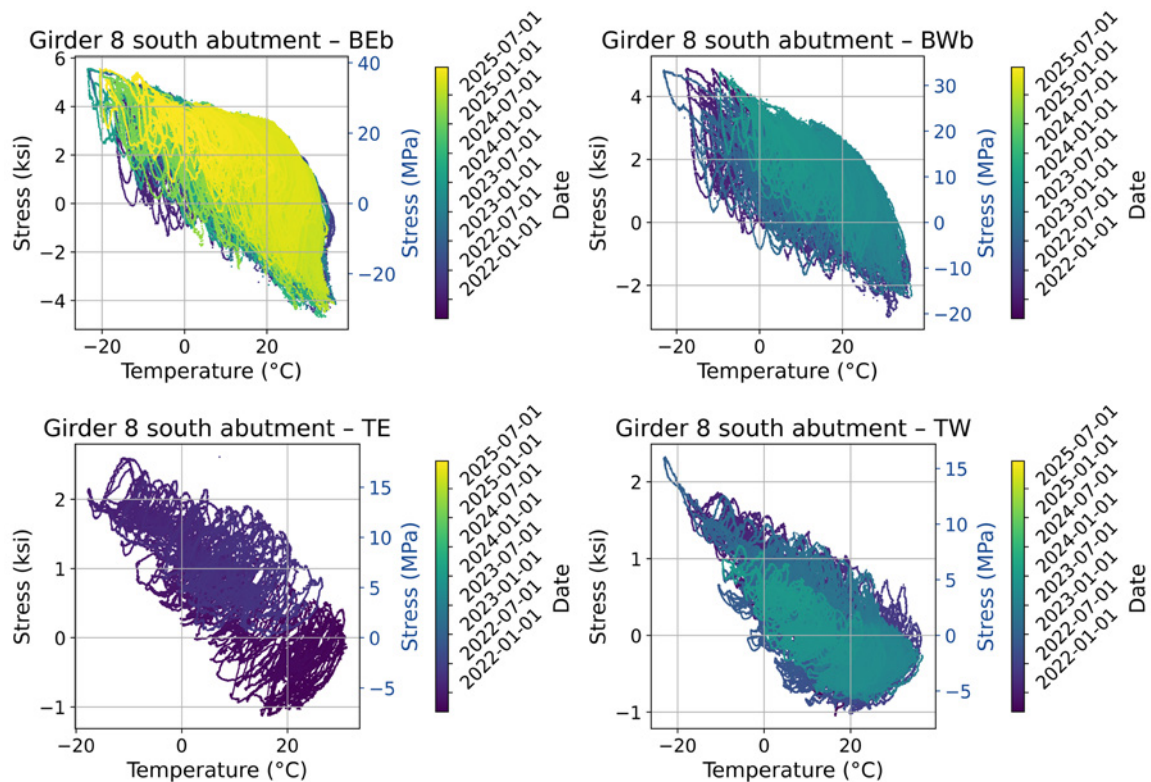


Figure 149. Graph. Temperature vs. stress data collected from Girder 8 South Abutment.

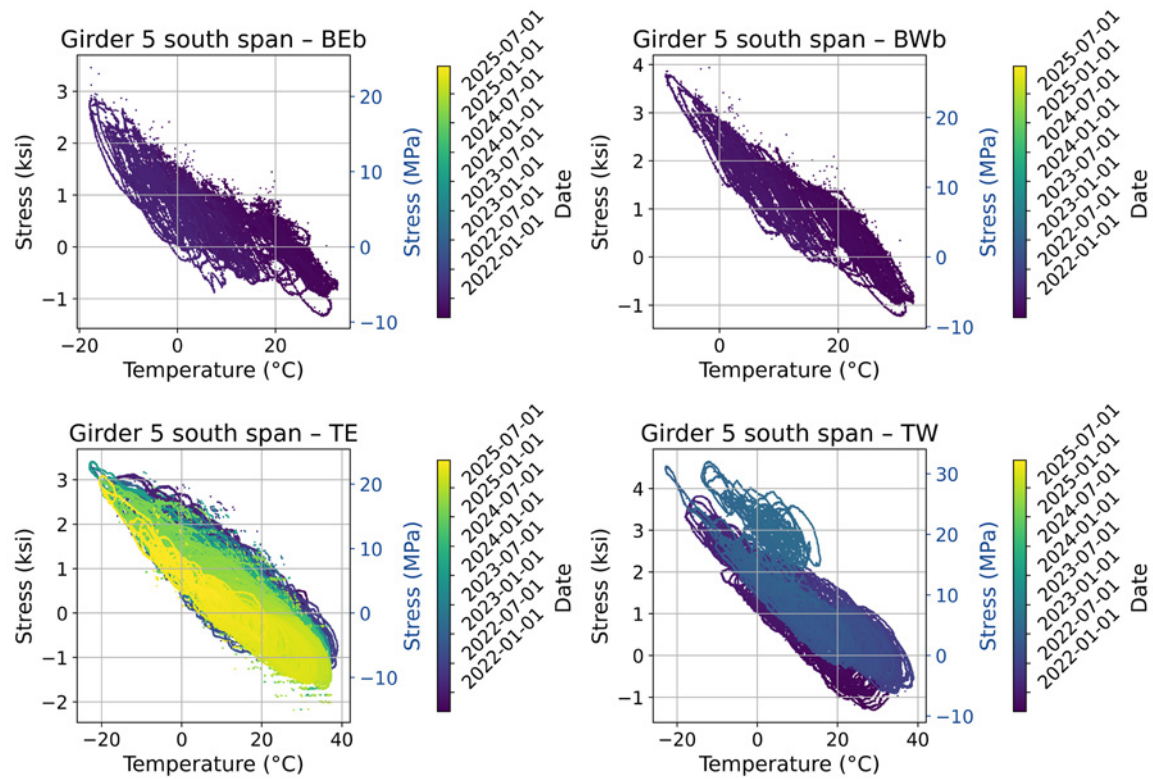


Figure 150. Graph. Temperature vs. stress data collected from Girder 5 South Span.

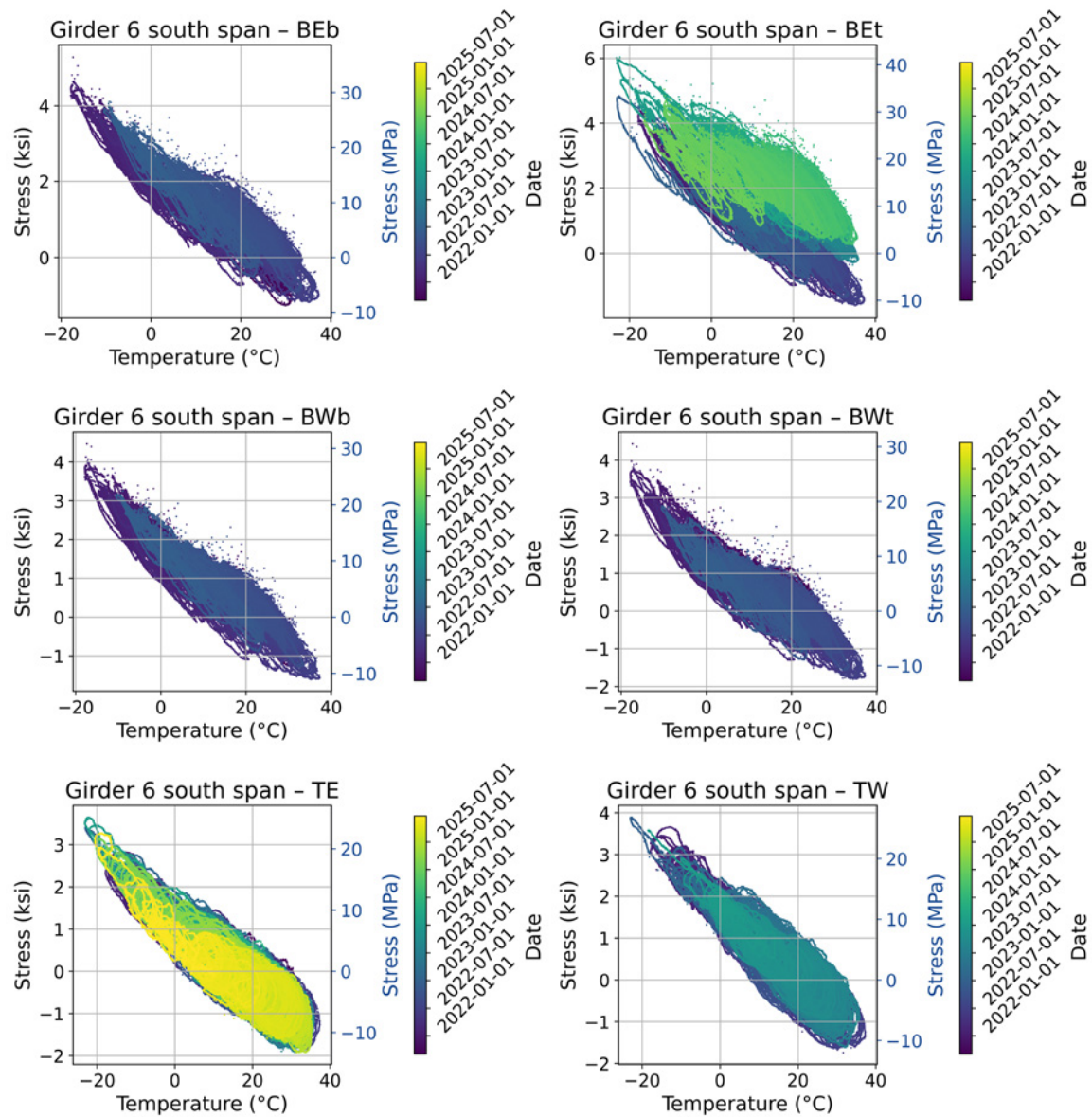


Figure 151. Graph. Temperature vs. stress data collected from Girder 6 South Span.

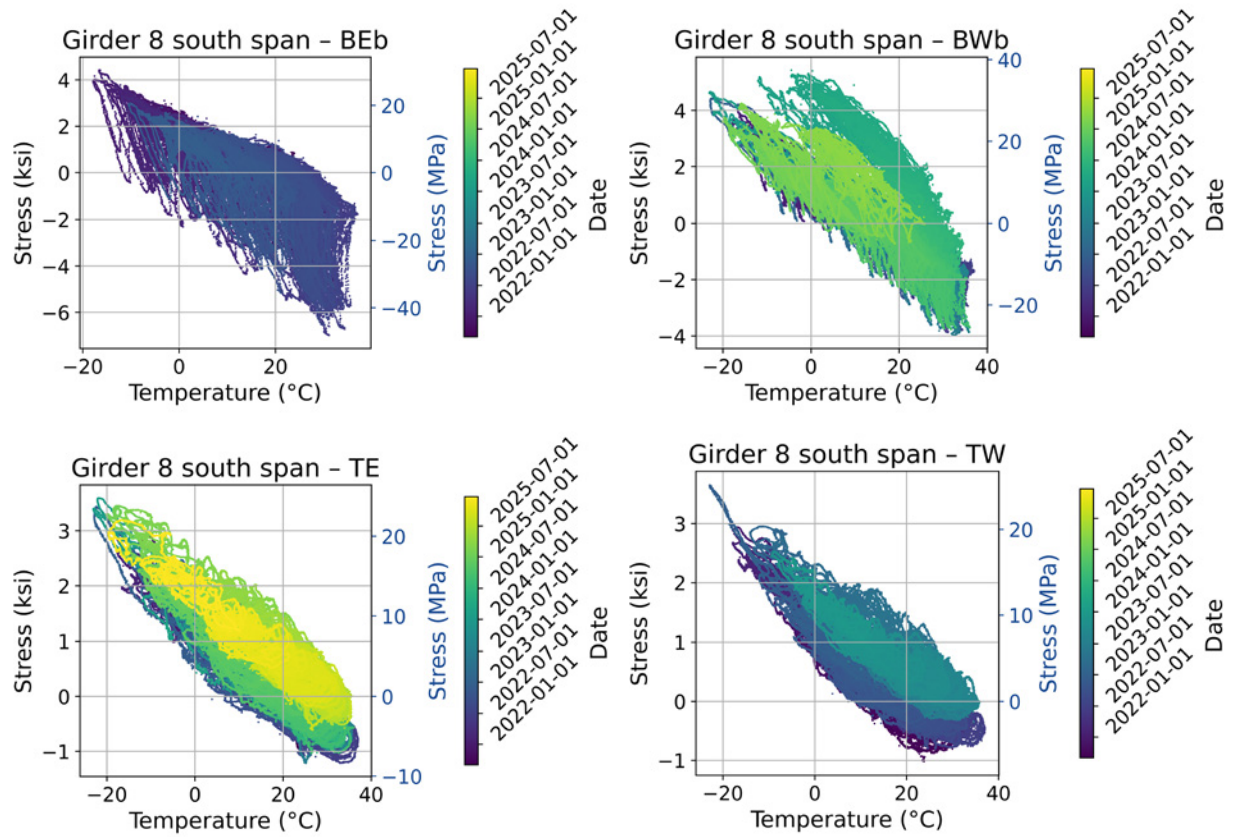


Figure 152. Graph. Temperature vs. stress data collected from Girder 8 South Span.

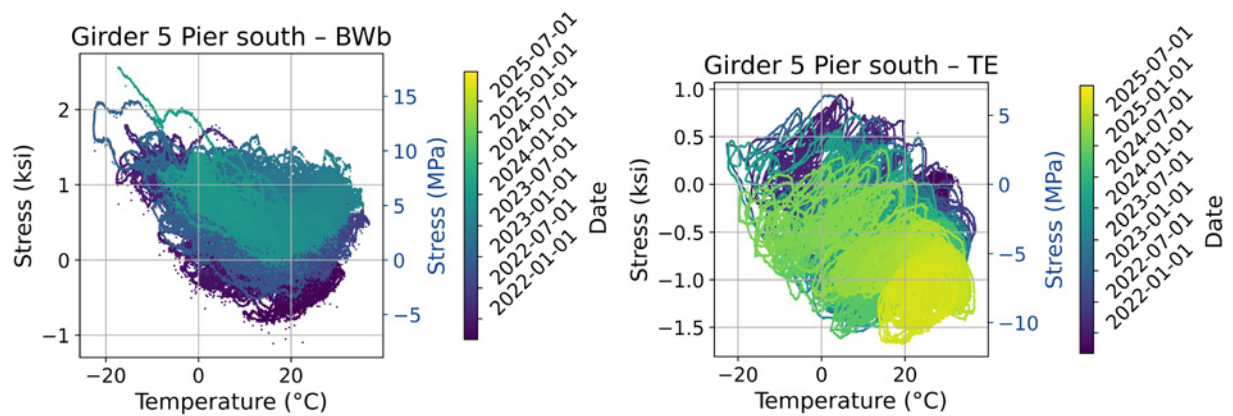


Figure 153. Graph. Temperature vs. stress data collected from Girder 5 Pier South.

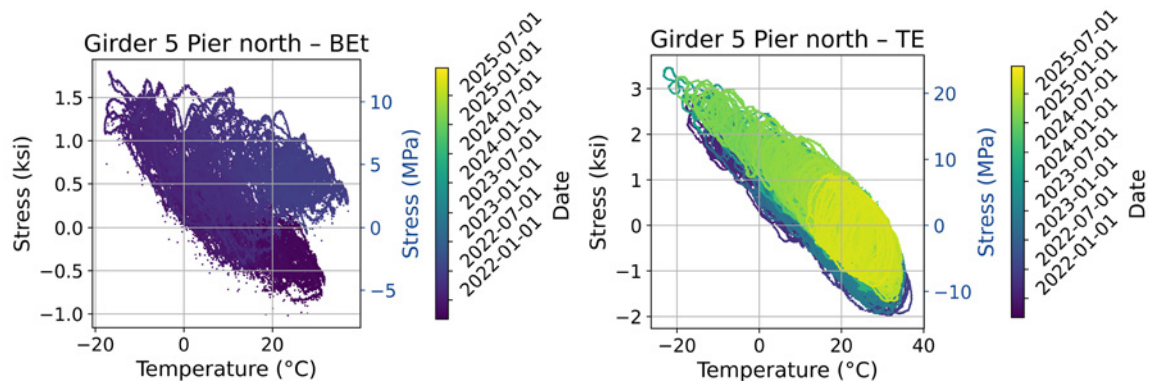


Figure 154. Graph. Temperature vs. stress data collected from Girder 5 Pier North.

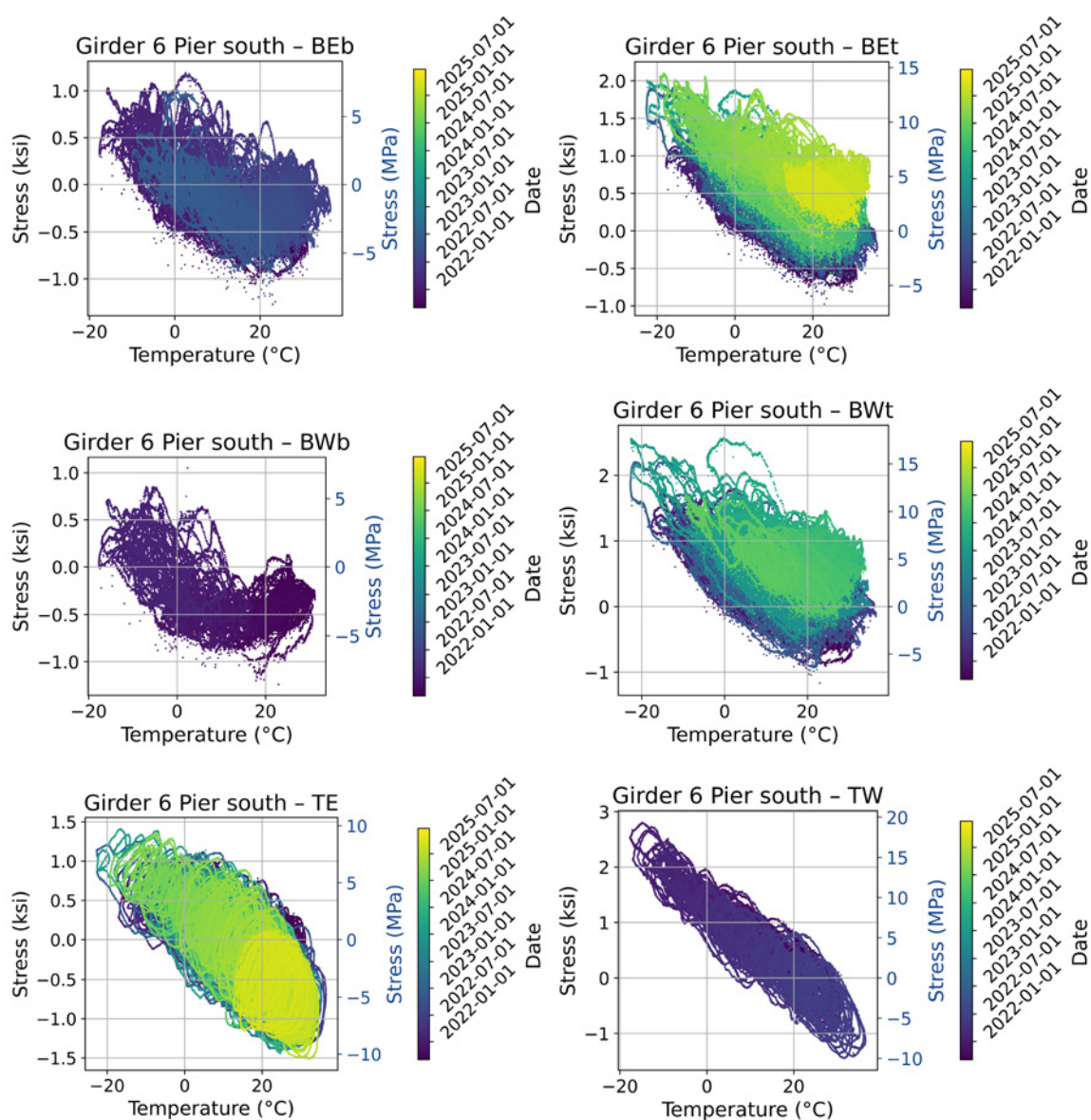


Figure 155. Graph. Temperature vs. stress data collected from Girder 6 Pier South.

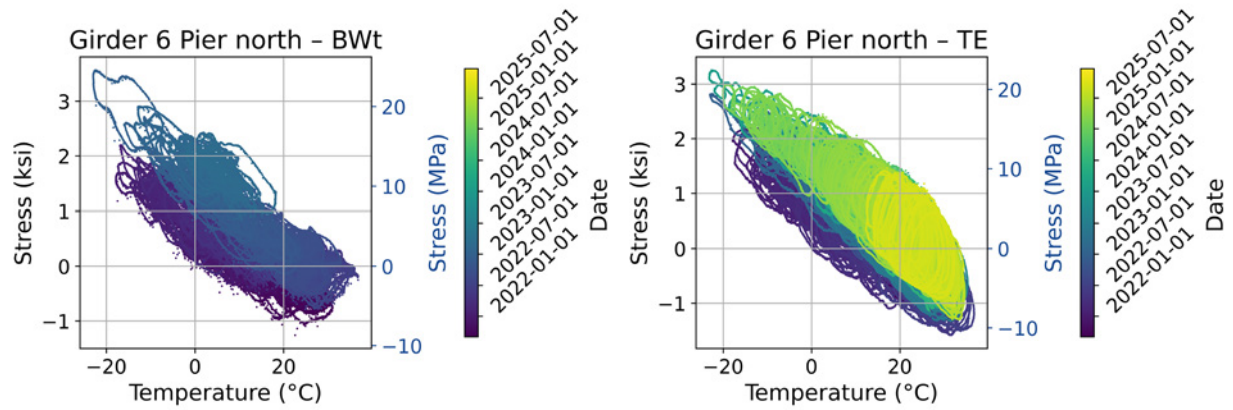


Figure 156. Graph. Temperature vs. stress data collected from Girder 6 Pier North.

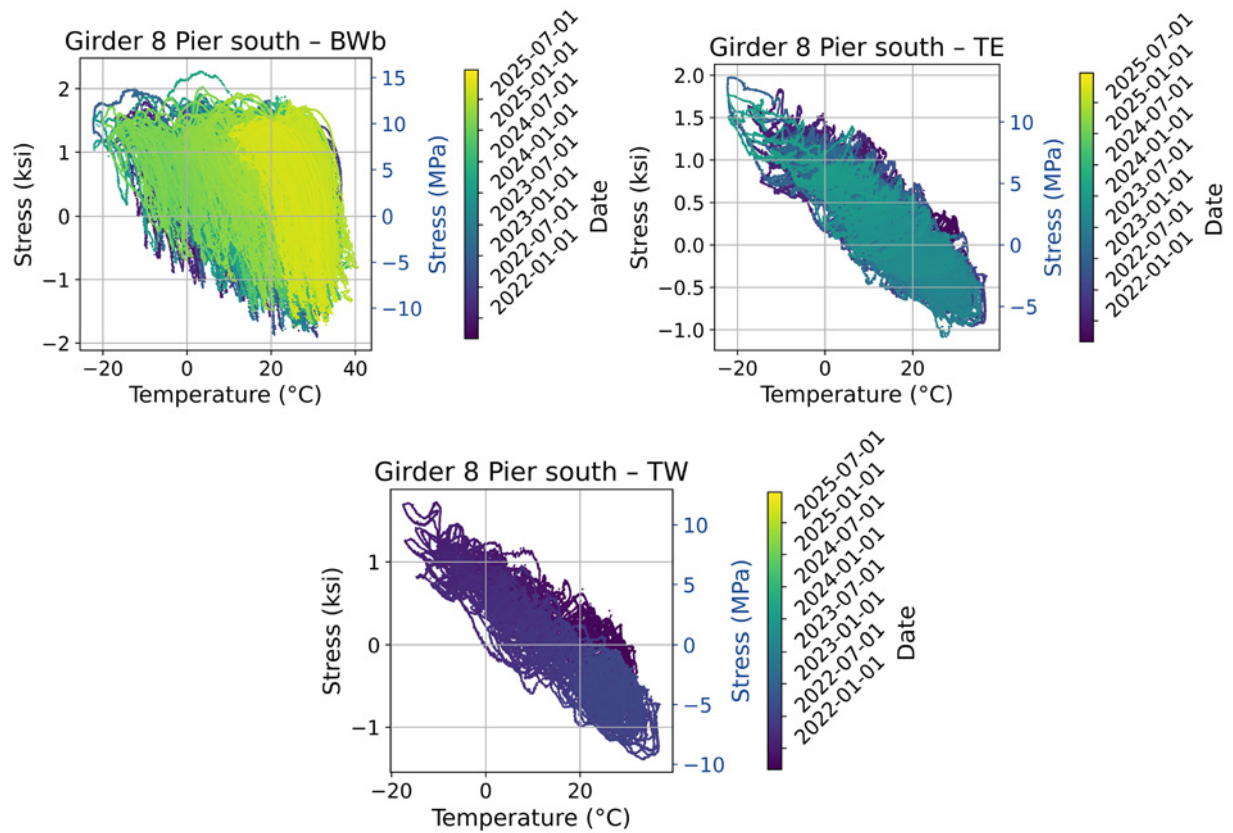


Figure 157. Graph. Temperature vs. stress data collected from Girder 8 Pier.

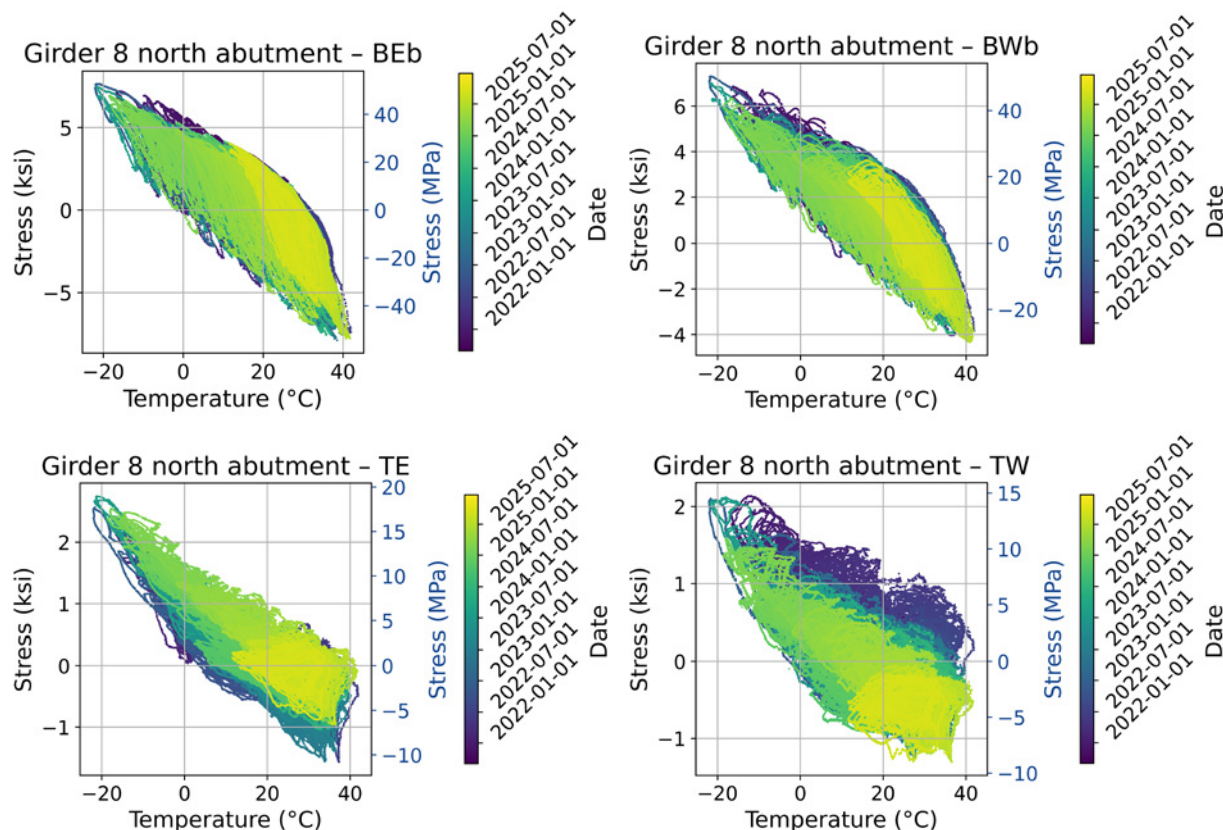


Figure 158. Graph. Temperature vs. stress data collected from Girder 8 North Abutment.

Decomposed Data from Girder Cross-Sections

Stress data obtained from girder cross-sections was decomposed into individual girder actions, including strong-axis bending, weak-axis bending, and axial stress, following the scheme presented in Volume I (Zhou et al. 2026b).

To provide an overview of these response components, Table 28 summarizes the key measures for all cross-sections, including maximum, minimum, and overall range of values over the monitoring period.

Table 28. Summary of Decomposed Stress Components from Girder Cross-Section Gauges—Mattis-57

Cross Section	Component	Min. Stress MPa (ksi)	Temp. at Min. (°C)	Max. Stress MPa (ksi)	Temp. at Max. (°C)	Stress Range MPa (ksi)
G5 S. Span	Axial	-6 (-0.9)	29.3	14.5 (2.1)	-8.1	20.5 (3)
G5 S. Span	SA bottom	-10.5 (-1.5)	12.8	14.5 (2.1)	19.8	25 (3.6)
G5 S. Span	SA top	-3 (-0.4)	19.8	2.5 (0.3)	12.8	5.5 (0.8)
G5 S. Span	WA bottom	-6 (-0.9)	-7.6	0.5 (0.1)	32.6	6.5 (1)
G5 S. Span	WA top	-1.5 (-0.2)	15.3	5 (0.7)	13.1	6.5 (1)
G6 Pier S.	Axial	-8 (-1.2)	33.5	12.5 (1.8)	-14.1	20.5 (2.9)
G6 Pier S.	SA bottom	-9.5 (-1.4)	19.7	9 (1.3)	32.5	18.5 (2.7)
G6 Pier S.	SA top	-1.5 (-0.2)	32.5	1.5 (0.2)	19.7	2.5 (0.4)
G6 Pier S.	WA bottom	-2 (-0.3)	4.2	1 (0.1)	20.2	3 (0.4)
G6 Pier S.	WA top	-6.5 (-1)	-14.3	2 (0.3)	33.4	8.5 (1.2)
G6 S. Abut.	Axial	-12.5 (-1.8)	34.7	23.5 (3.4)	-22.8	36 (5.2)
G6 S. Abut.	SA bottom	-18.5 (-2.7)	31.4	14.5 (2.1)	-2.8	33 (4.8)
G6 S. Abut.	SA top	-1.5 (-0.3)	-2.8	2 (0.3)	31.4	4 (0.6)
G6 S. Abut.	WA bottom	-5 (-0.7)	22.1	2.5 (0.4)	-22.7	7.5 (1.1)
G6 S. Abut.	WA top	-2 (-0.3)	19.8	2 (0.3)	25.8	4.5 (0.6)
G6 S. Span	Axial	-10.5 (-1.5)	34.7	22.5 (3.3)	-17.4	33 (4.8)
G6 S. Span	SA bottom	-9.5 (-1.4)	19.1	18 (2.5)	16.4	27 (3.9)
G6 S. Span	SA top	-4.5 (-0.6)	16.4	2.5 (0.3)	19.1	7 (1)
G6 S. Span	WA bottom	0 (0)	29.2	3 (0.4)	-2.5	3 (0.4)
G6 S. Span	WA top	-4.5 (-0.6)	1.1	1 (0.2)	29.1	5.5 (0.8)
G7 S. Abut.	Axial	-10 (-1.5)	29.3	17 (2.5)	-10.5	27.5 (4)
G7 S. Abut.	SA bottom	-23.5 (-3.4)	31.2	18.5 (2.7)	-8.0	42 (6.1)
G7 S. Abut.	SA top	-1 (-0.1)	-8.0	1 (0.2)	31.2	2 (0.3)
G7 S. Abut.	WA bottom	-3.5 (-0.5)	33.7	4.5 (0.6)	-12.2	8 (1.2)
G7 S. Abut.	WA top	-5 (-0.7)	24.5	1 (0.1)	7.2	5.5 (0.8)
G8 N. Abut.	Axial	-13 (-1.9)	41.5	25.5 (3.7)	-21.8	38.5 (5.6)
G8 N. Abut.	SA bottom	-29 (-4.2)	37.4	28 (4.1)	-14.1	57 (8.3)
G8 N. Abut.	SA top	-10.5 (-1.6)	-14.1	11 (1.6)	37.4	22 (3.2)
G8 N. Abut.	WA bottom	-19 (-2.8)	30.5	10.5 (1.5)	20.2	29.5 (4.3)
G8 N. Abut.	WA top	-4 (-0.6)	0.8	4.5 (0.6)	-11.8	8.5 (1.2)
G8 S. Abut.	Axial	-6.5 (-1)	16.5	15.5 (2.2)	-11.1	22 (3.2)
G8 S. Abut.	SA bottom	-19 (-2.8)	16.9	25 (3.6)	-8.2	44 (6.4)
G8 S. Abut.	SA top	-1 (-0.1)	-8.2	1 (0.1)	16.9	2 (0.3)
G8 S. Abut.	WA bottom	-9 (-1.3)	21.1	4 (0.6)	16.8	13 (1.9)
G8 S. Abut.	WA top	-1.5 (-0.2)	7.5	6.5 (0.9)	7.2	8 (1.1)
G8 S. Span	Axial	-10 (-1.4)	35.3	21.5 (3.1)	-17.3	31.5 (4.5)
G8 S. Span	SA bottom	-29 (-4.2)	31.0	12 (1.8)	3.9	41 (6)
G8 S. Span	SA top	-3 (-0.5)	3.9	7.5 (1.1)	31.0	11 (1.6)
G8 S. Span	WA bottom	-15 (-2.2)	29.0	2.5 (0.3)	23.8	18 (2.5)
G8 S. Span	WA top	-2.5 (-0.4)	1.9	2.5 (0.3)	6.7	5 (0.7)

All data corresponding to the summary presented in Table 28 is illustrated in the response history plots below (Figure 159 through Figure 166), which shows the full temporal evolution of each decomposed stress component per girder cross-section.

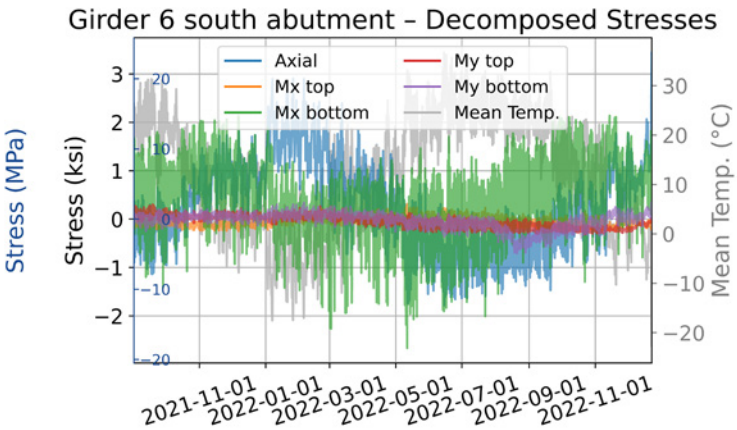


Figure 159. Graph. Decomposed stress data collected from Girder 6 South Abutment.

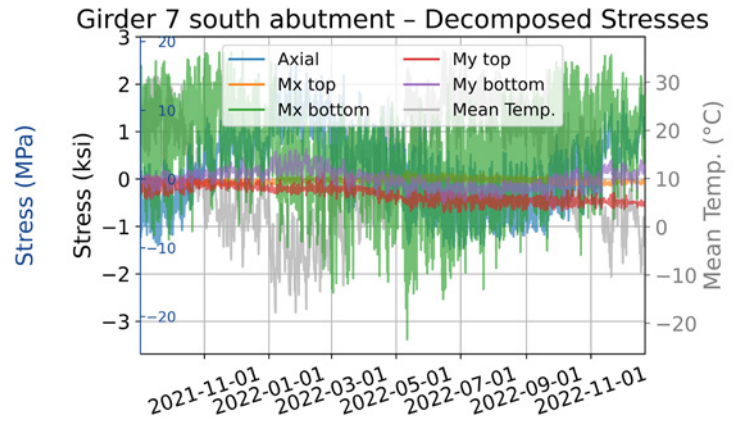


Figure 160. Graph. Decomposed stress data collected from Girder 7 South Abutment.

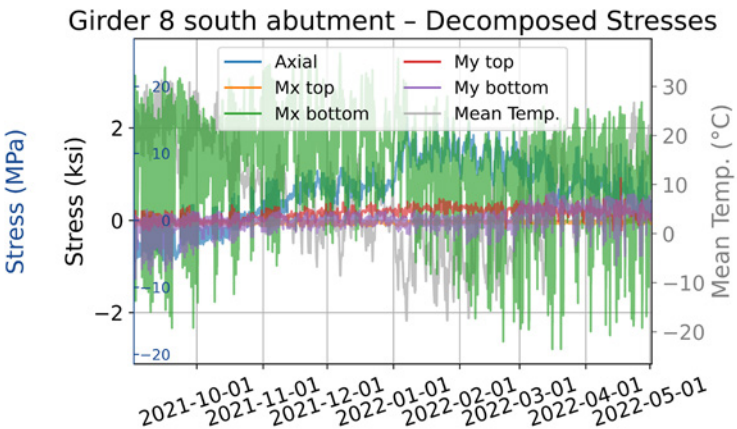


Figure 161. Graph. Decomposed stress data collected from Girder 8 South Abutment.

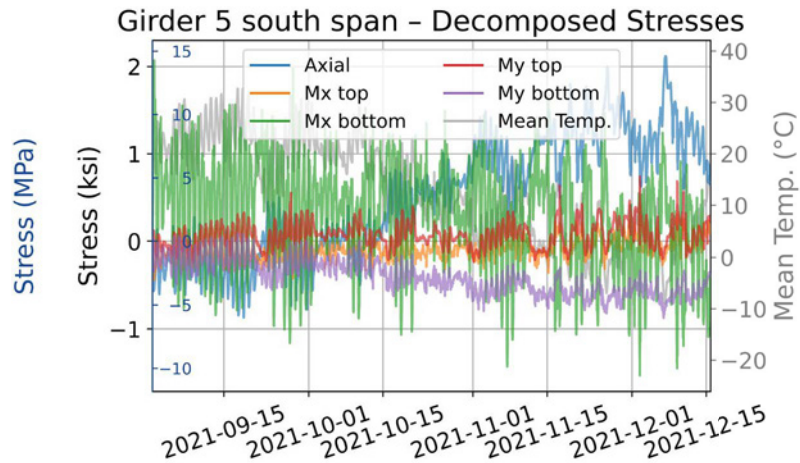


Figure 162. Graph. Decomposed stress data collected from Girder 5 South Span.

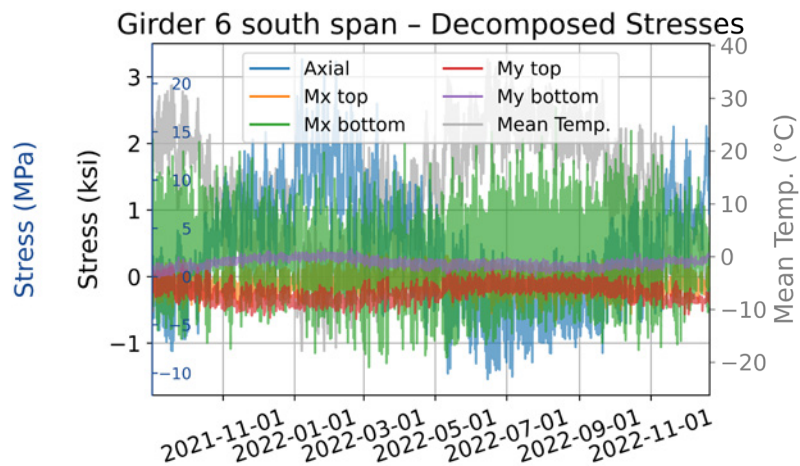


Figure 163. Graph. Decomposed stress data collected from Girder 6 South Span.

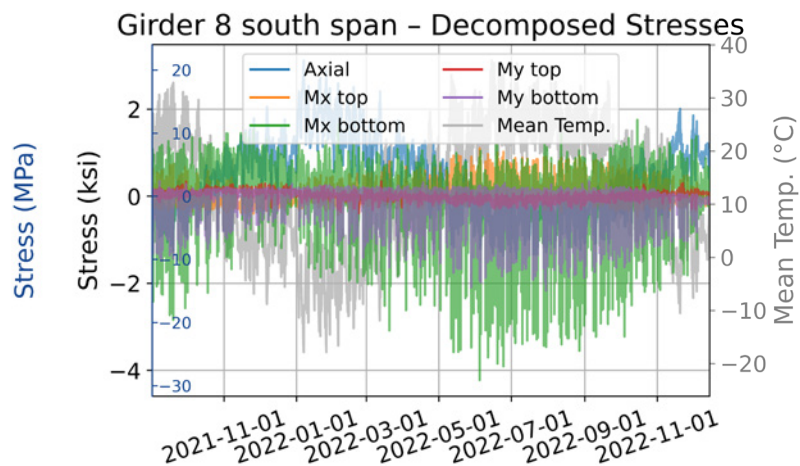


Figure 164. Graph. Decomposed stress data collected from Girder 8 South Span.

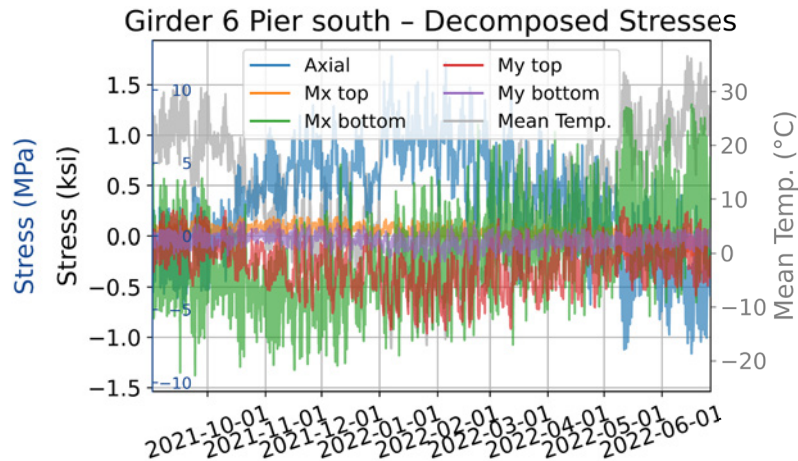


Figure 165. Graph. Decomposed stress data collected from Girder 6 Pier South.

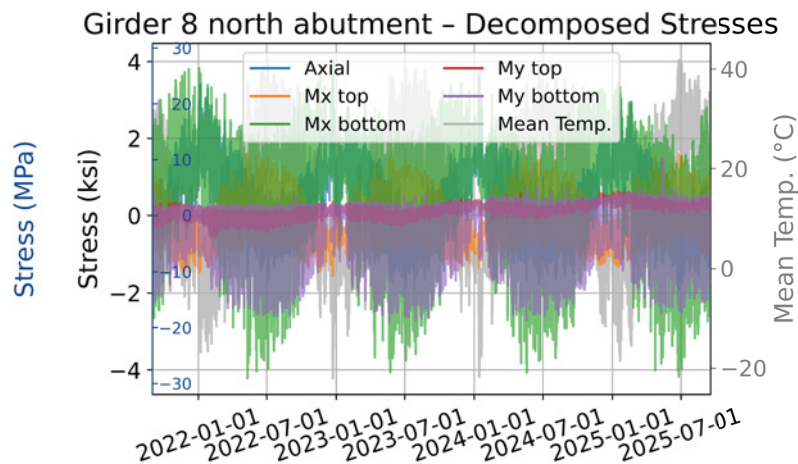


Figure 166. Graph. Decomposed stress data collected from Girder 8 North Abutment.

The same decomposed stress data presented above, is presented in temperature versus decomposed stress readings in the plots below (Figure 167 through Figure 174). The temperature used for these plots is the average temperature collected from thermistors attached to all gauges mounted on the cross-section.

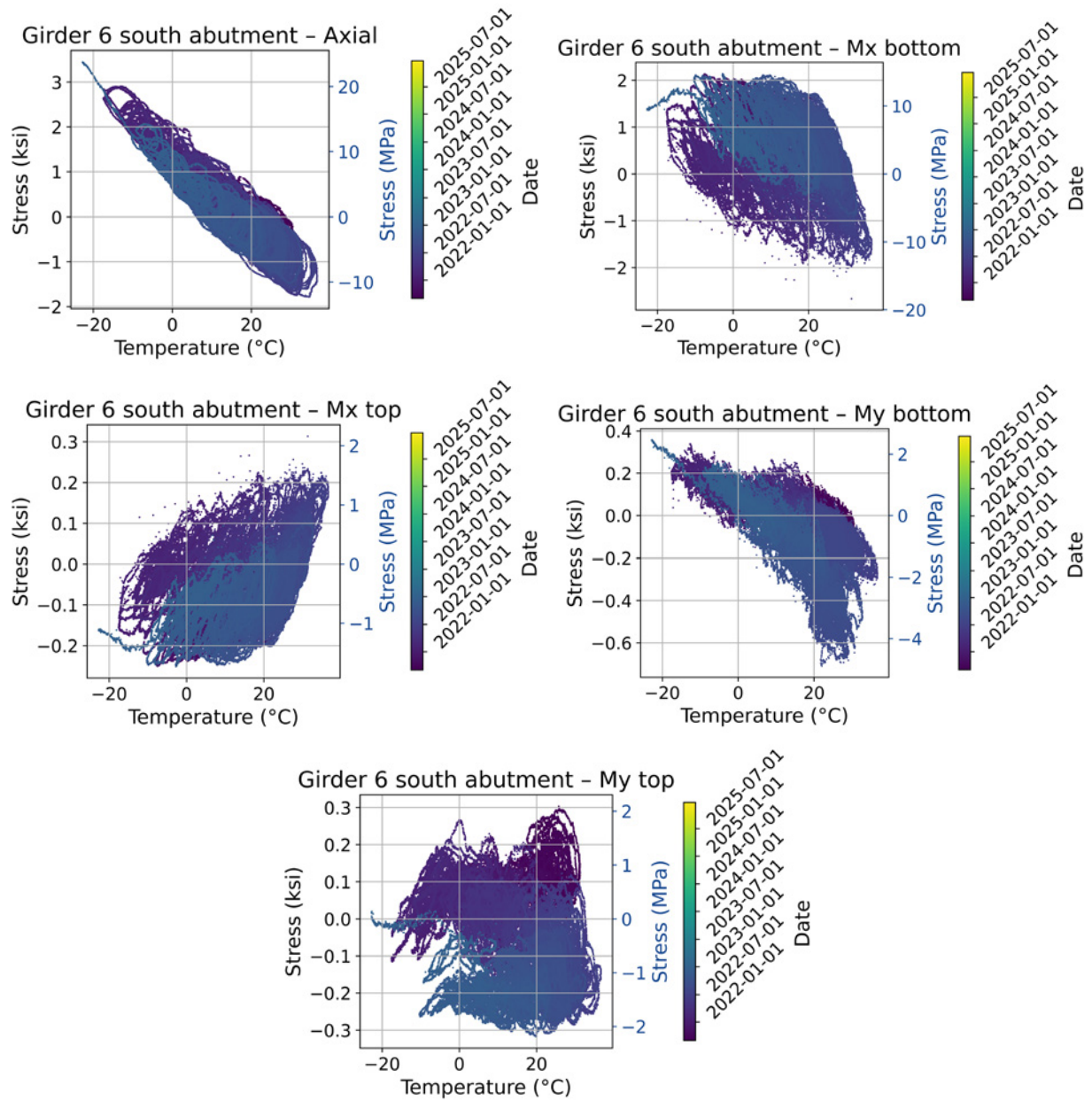


Figure 167. Graph. Temperature vs. decomposed stress data collected from Girder 6 South Abutment.

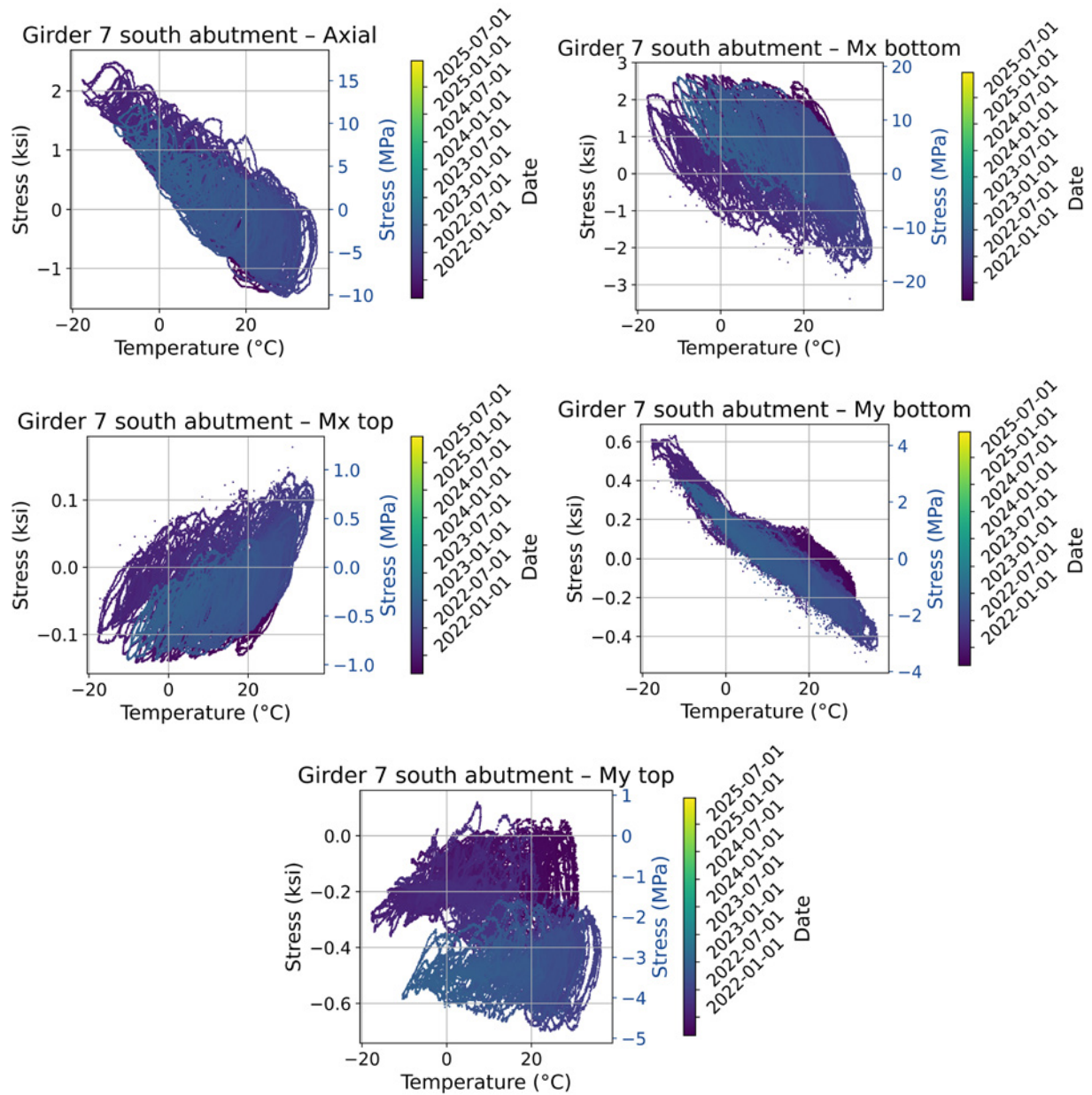


Figure 168. Graph. Temperature vs. decomposed stress data collected from Girder 7 South Abutment.

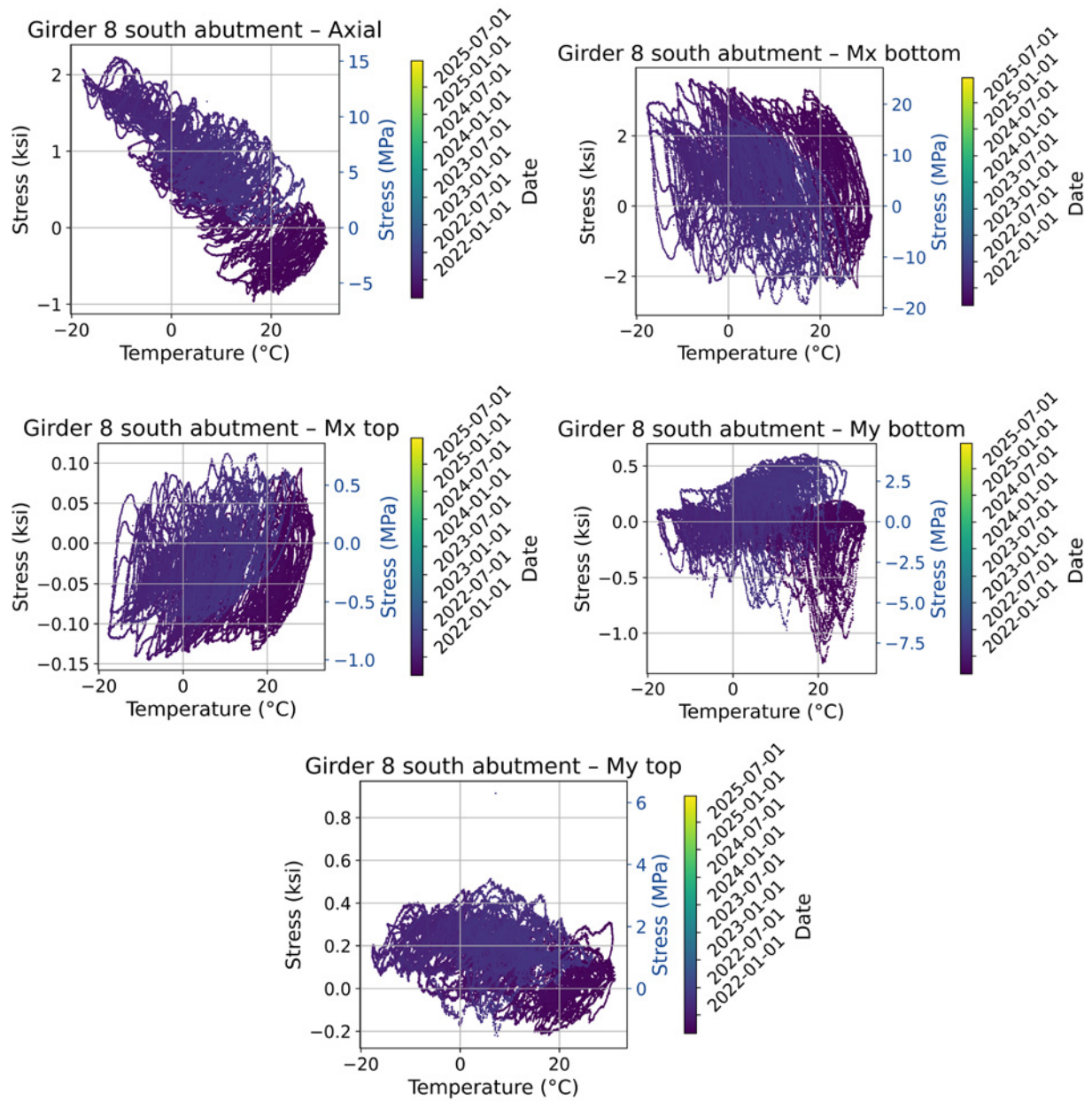


Figure 169. Graph. Temperature vs. decomposed stress data collected from Girder 8 South Abutment.

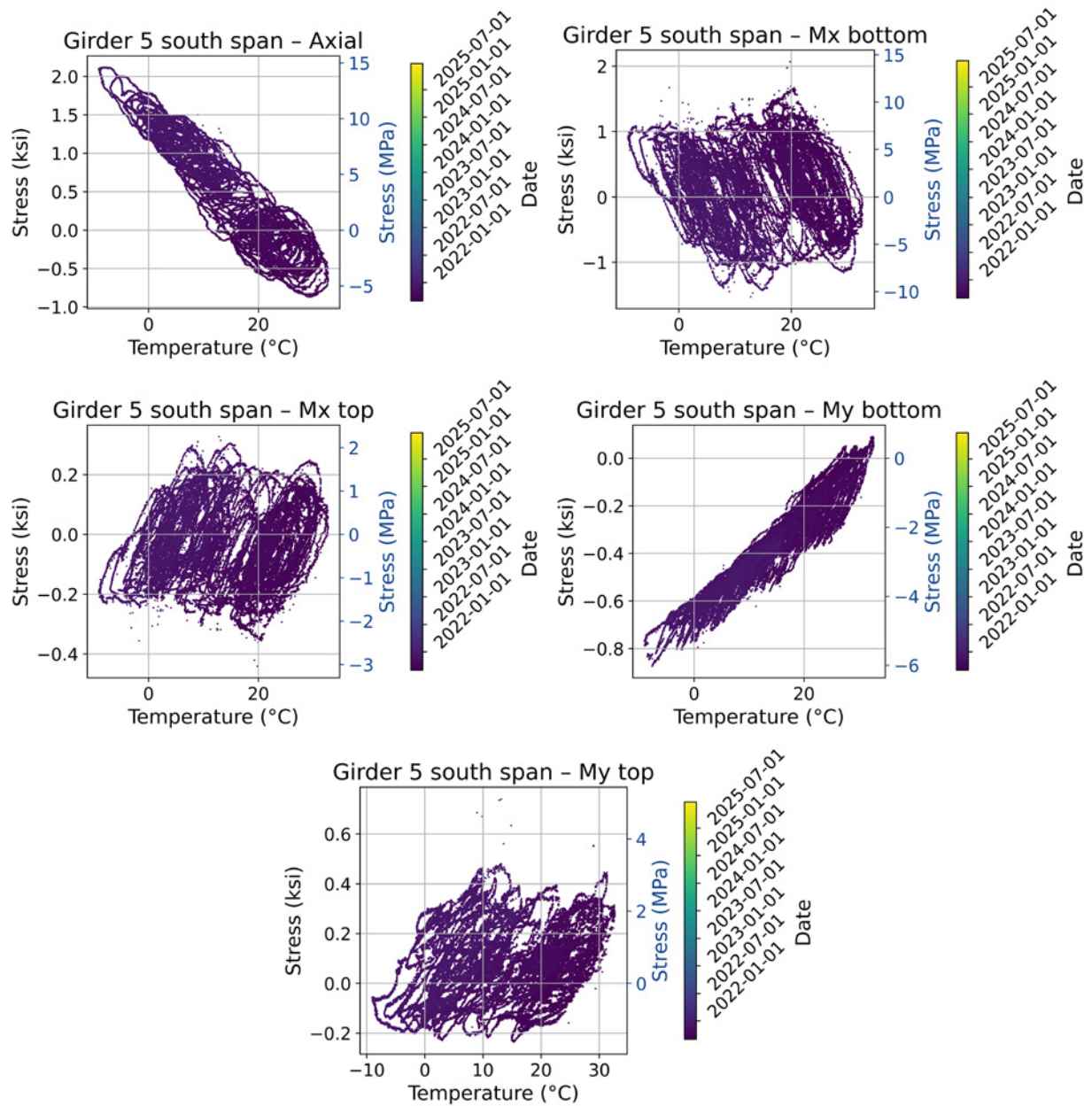


Figure 170. Graph. Temperature vs. decomposed stress data collected from Girder 5 South Span.

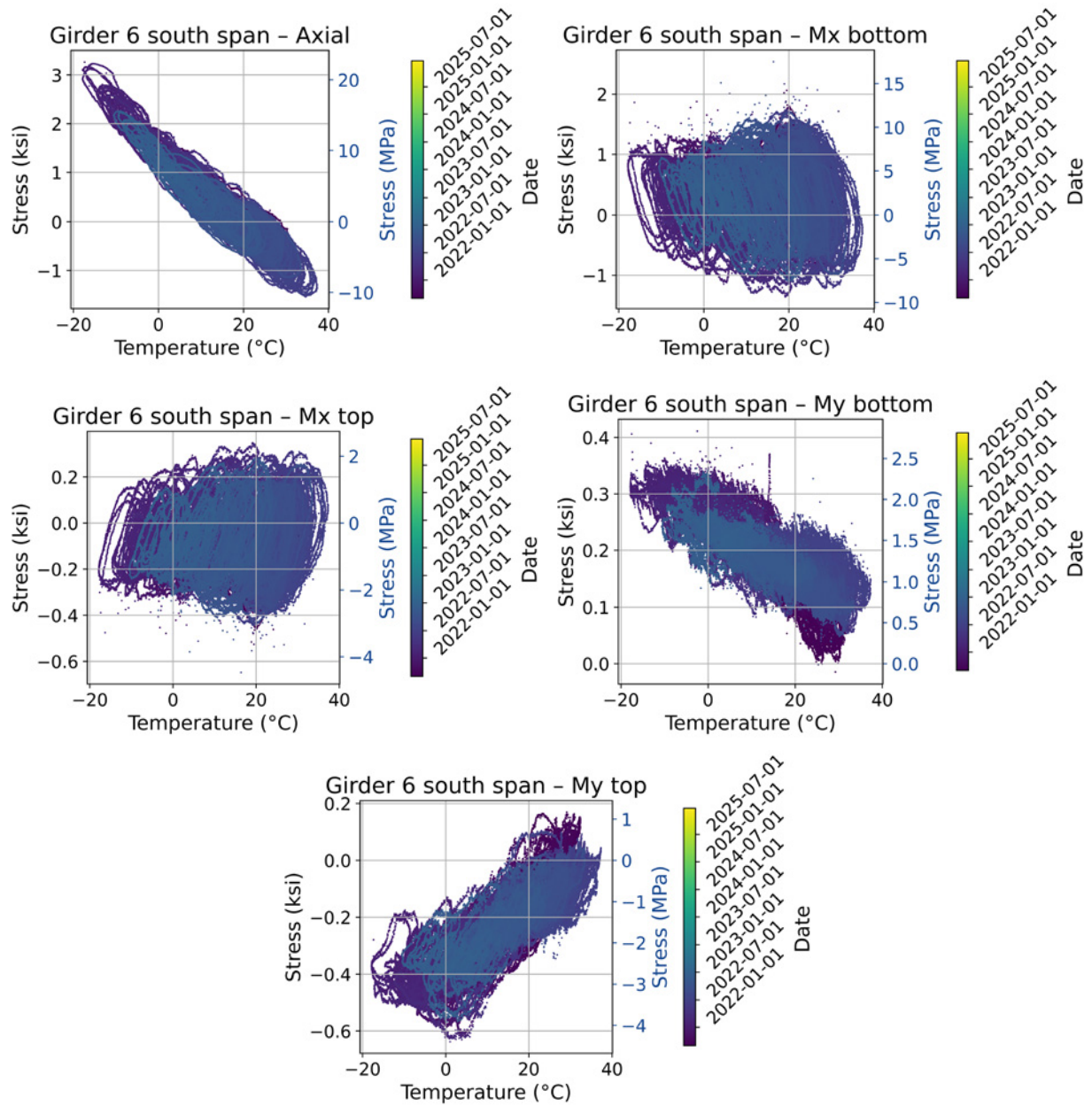


Figure 171. Graph. Temperature vs. decomposed stress data collected from Girder 6 South Span.

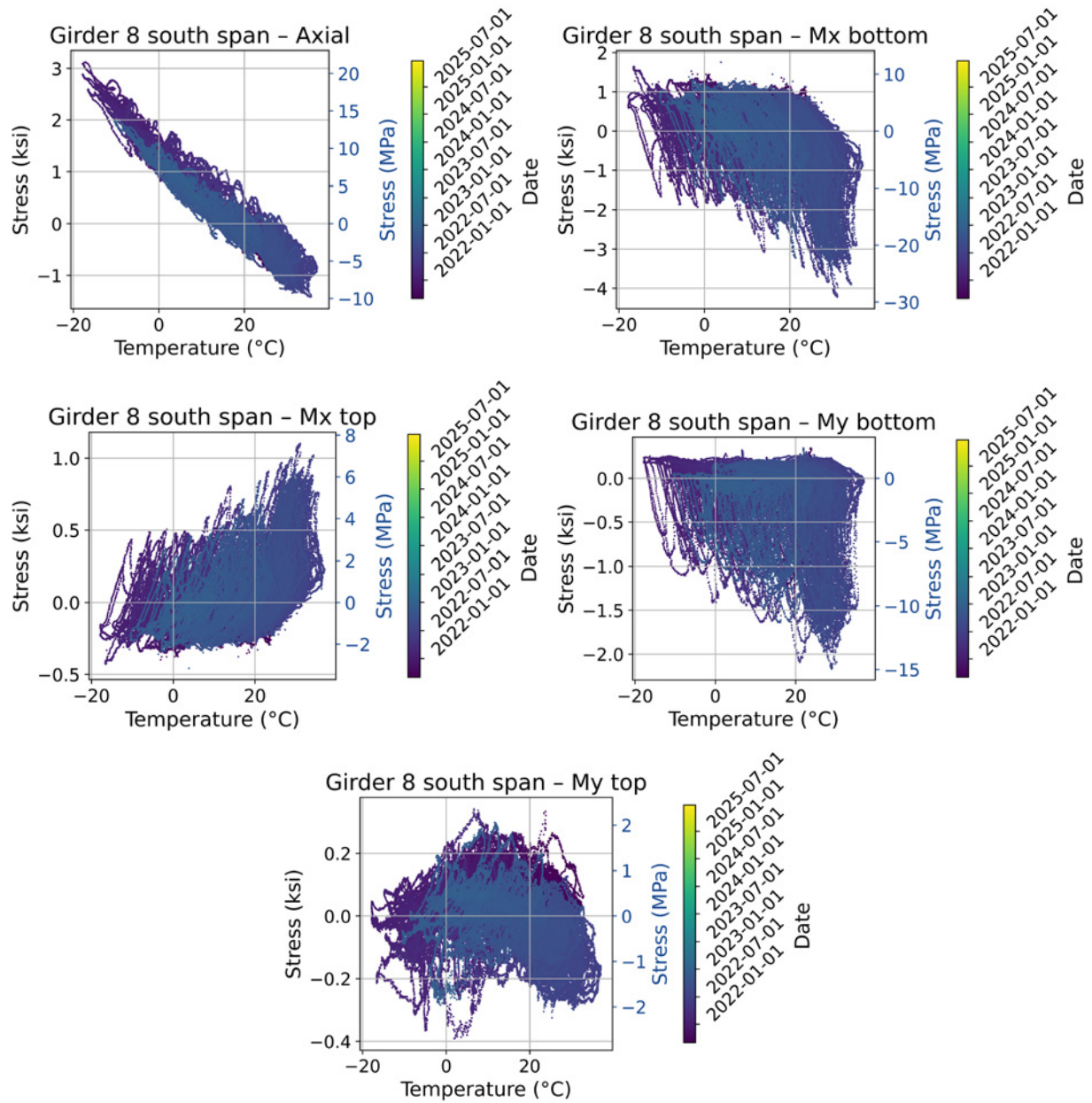


Figure 172. Graph. Temperature vs. decomposed stress data collected from Girder 8 South Span.

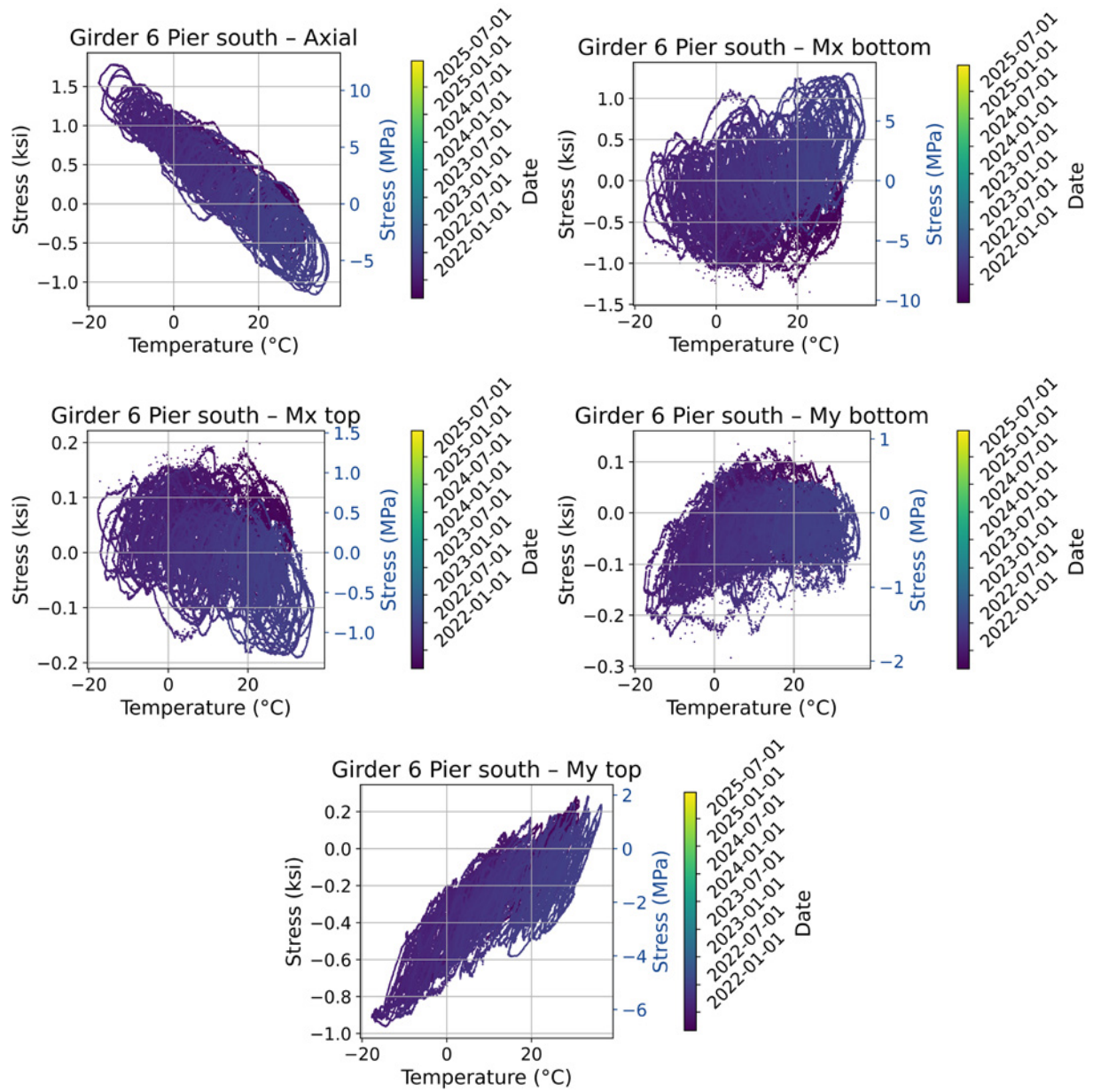


Figure 173. Graph. Temperature vs. decomposed stress data collected from Girder 6 Pier South.

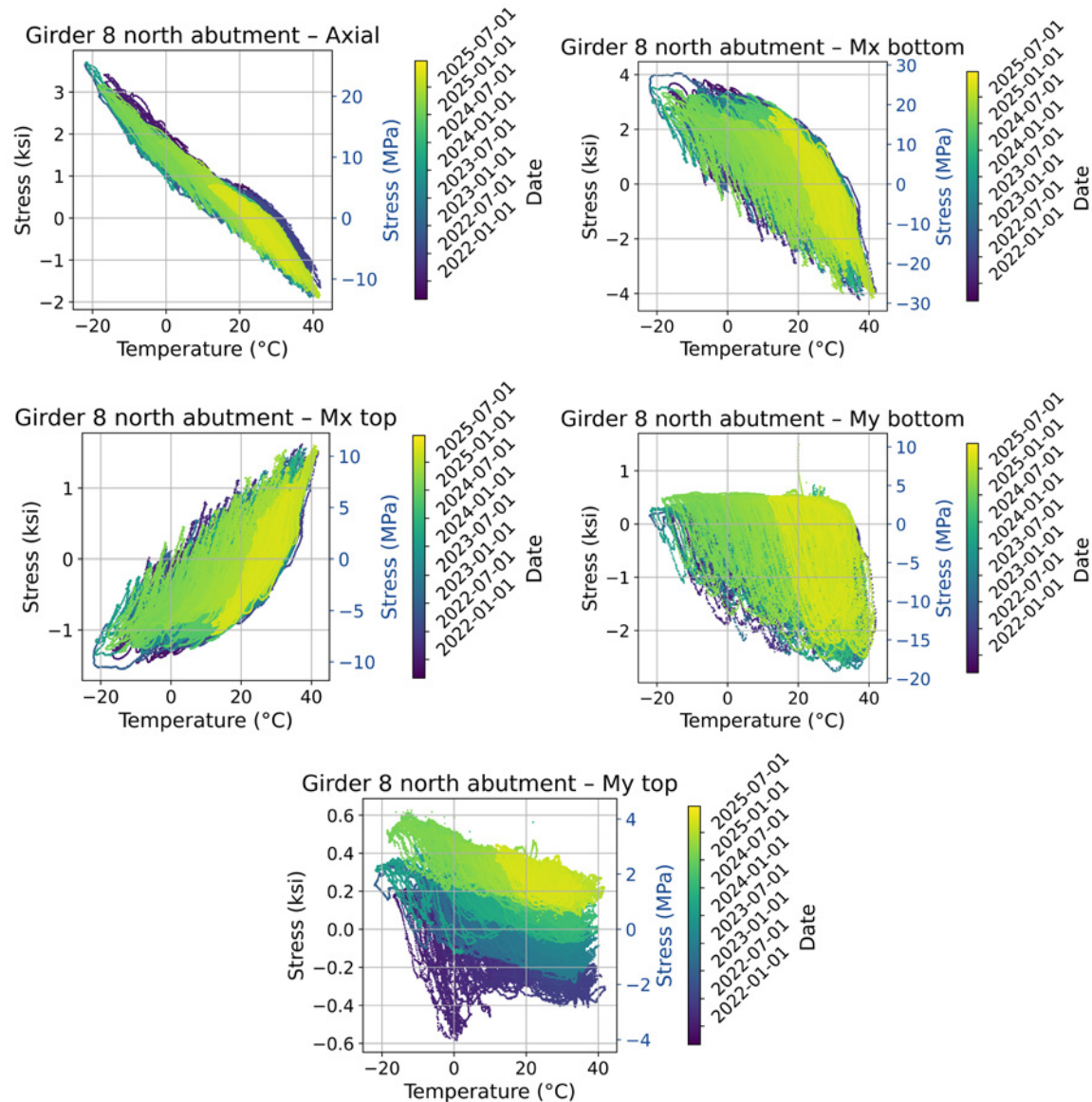


Figure 174. Graph. Temperature vs. decomposed stress data collected from Girder 8 North Abutment.

Cross-Frames

A total of 15 gauges were mounted on 5 different steel cross-frames. Three strain gauges were typically installed at an instrumented cross-frame location: at the west top portion of one diagonal angle (W), at the east top portion of the other diagonal angle (E), and at mid-length of the bottom chord angle on the concentric leg (Bc) (Figure 3).

Unlike the girder instrumentation discussed previously, readings exhibiting potential strain gauge malfunction were not removed from the cross-frame dataset. Consequently, some plots (and summary tables) in this section may include localized deviations or long-term drift behavior that are

not necessarily representative of the actual structural response, but are retained herein to provide a complete representation of the recorded measurements.

To provide an overview of the recorded response, Table 29 summarizes the key measures for all cross-frame gauges, including maximum, minimum, and overall range of stress values over the monitoring period.

Table 29. Summary of Stress Data from Cross-Frame Gauges—Mattis-57

Cross Frame	Member	Min. Stress MPa (ksi)	Temp. at Min. (°C)	Max. Stress MPa (ksi)	Temp. at Max. (°C)	Stress Range MPa (ksi)
CF1 5&6	B	-11 (-1.6)	31.9	15.5 (2.2)	-20.0	26 (3.8)
CF1 5&6	E	-12.5 (-1.8)	22.9	14.5 (2.1)	-16.2	27 (3.9)
CF1 5&6	W	-20 (-2.9)	31.9	37.5 (5.4)	-20.1	57.5 (8.3)
CF1 7&8	E	-16.5 (-2.4)	23.6	12 (1.7)	-16.7	28 (4.1)
CF1 7&8	W	-16.5 (-2.4)	23.7	28 (4.1)	-22.5	44.5 (6.4)
CF17 7&8	B	-5.5 (-0.8)	9.7	20 (2.9)	12.6	25 (3.6)
CF17 7&8	E	-6.5 (-0.9)	29.2	30 (4.3)	-14.3	36.5 (5.3)
CF4 5&6	B	-10.5 (-1.5)	22.7	11 (1.6)	13.2	21 (3.1)
CF4 5&6	E	-9 (-1.3)	15.9	15.5 (2.2)	19.7	24 (3.5)
CF4 5&6	W	-3 (-0.4)	22.1	34 (4.9)	-10.5	37 (5.3)
CF8 5&6	B	-5.5 (-0.8)	25.7	11.5 (1.7)	-22.0	17 (2.4)
CF8 5&6	E	-8.5 (-1.2)	27.8	24.5 (3.6)	-23.2	33 (4.8)
CF8 5&6	W	-10.5 (-1.5)	23.7	15.5 (2.2)	-9.3	25.5 (3.7)

All data corresponding to the summary presented in Table 29 is illustrated in the response history plots below (Figure 175 through Figure 179), which shows the full temporal evolution of the recorded stresses.

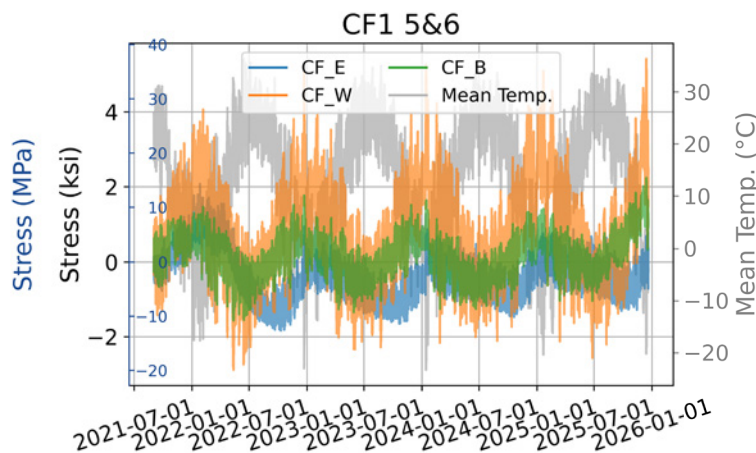


Figure 175. Graph. Stress data collected from CF1 5&6.

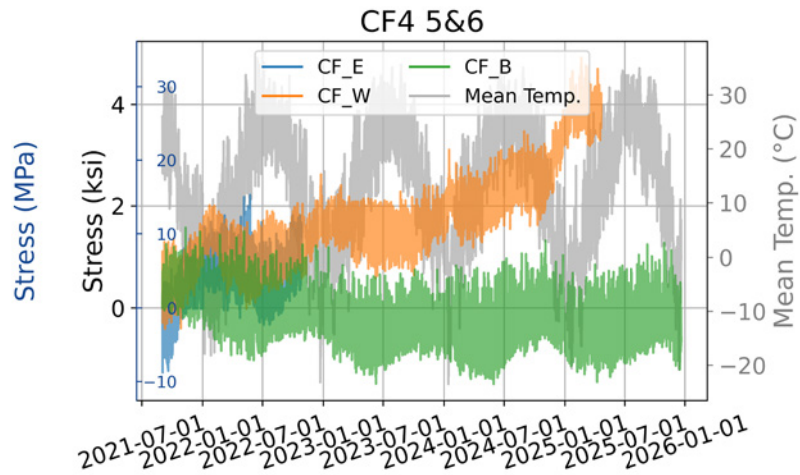


Figure 176. Graph. Stress data collected from CF4 5&6.

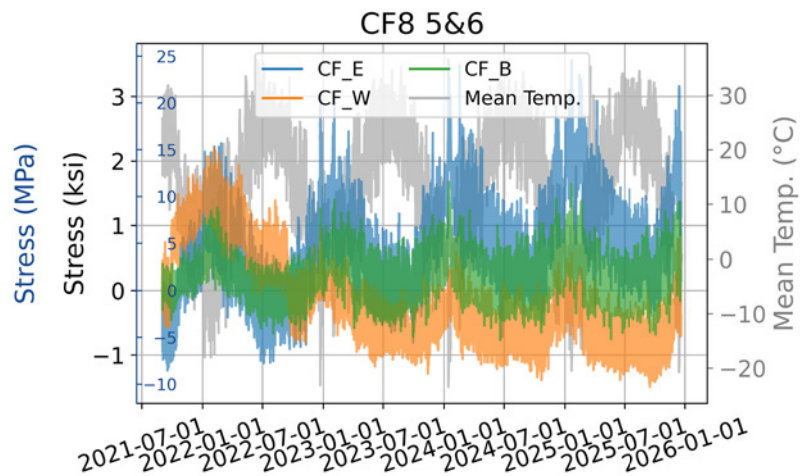


Figure 177. Graph. Stress data collected from CF8 5&6.

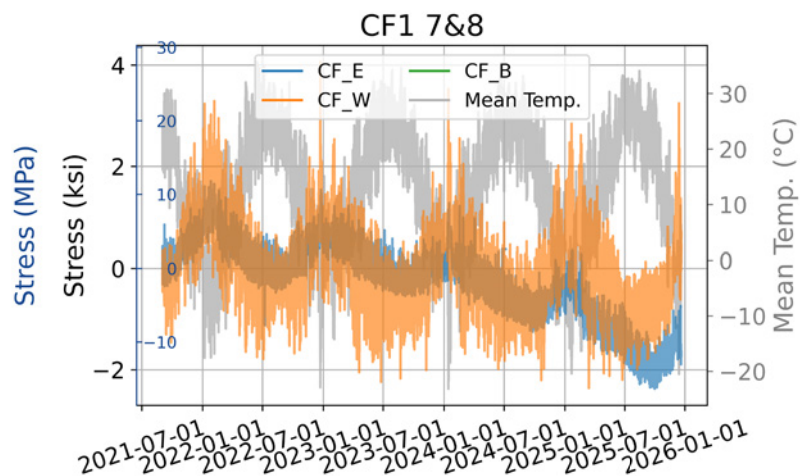


Figure 178. Graph. Stress data collected from CF1 7&8.

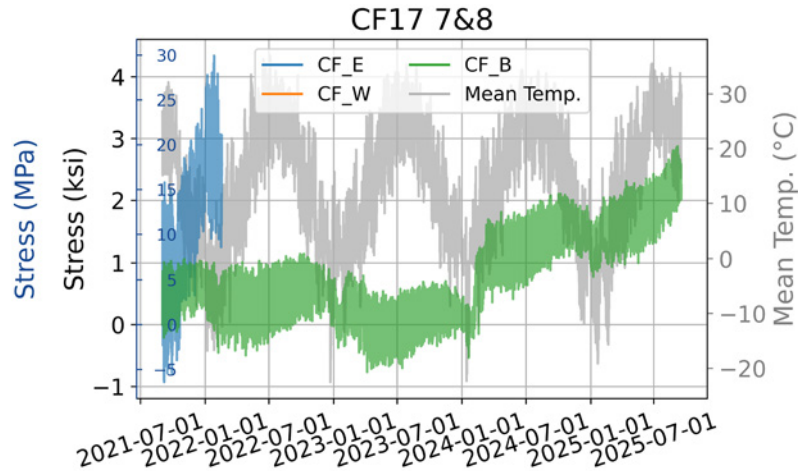


Figure 179. Graph. Stress data collected from CF17 7&8.

The same stress data presented above, is presented in temperature versus decomposed stress readings in the plots below (Figure 180 through Figure 184). The temperature used for these plots is the average temperature collected from thermistors attached to all gauges mounted on the cross-frame.

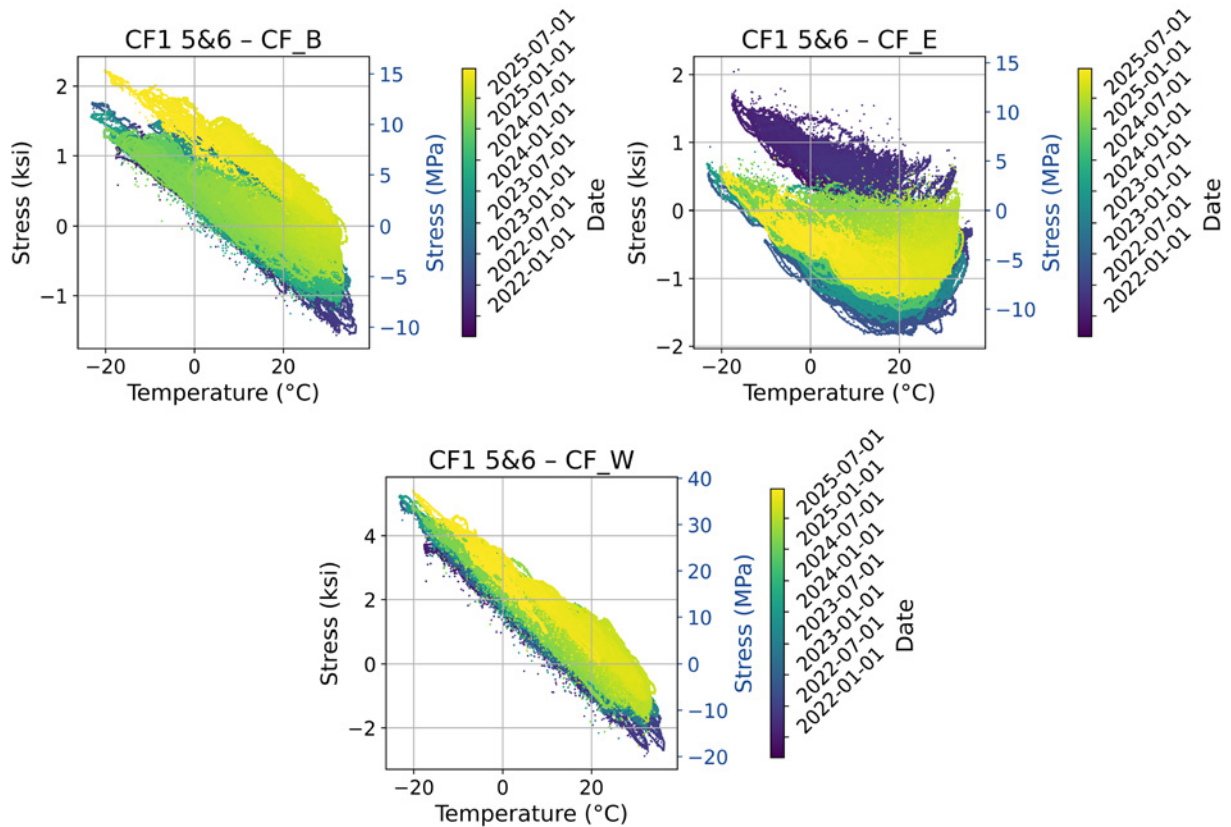


Figure 180. Graph. Temperature vs. stress data collected from CF1 5&6.

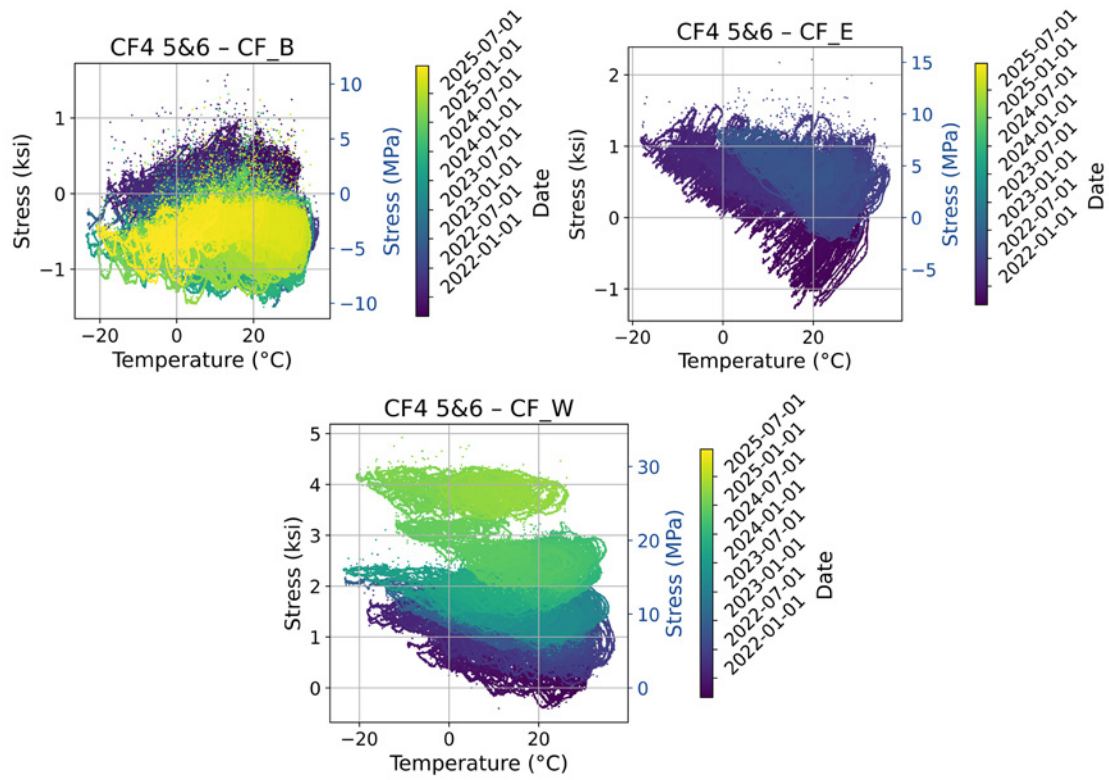


Figure 181. Graph. Temperature vs. stress data collected from CF4 5&6.

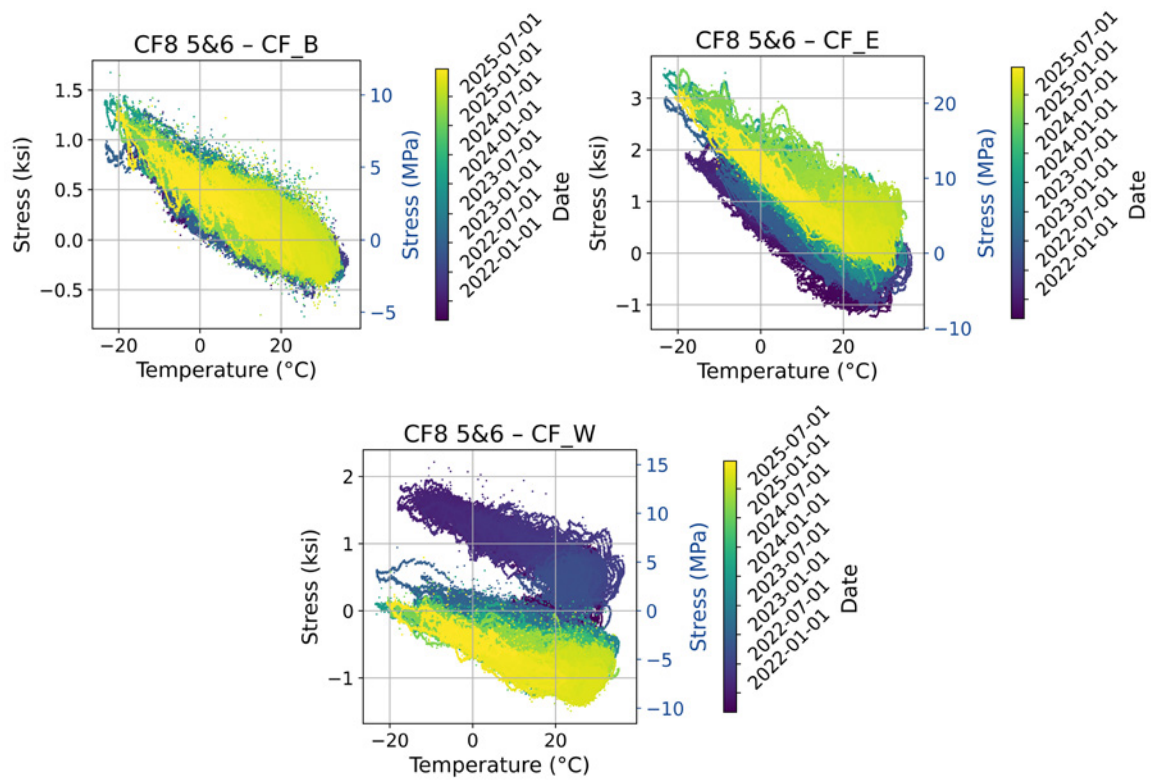


Figure 182. Graph. Temperature vs. stress data collected from CF8 5&6.

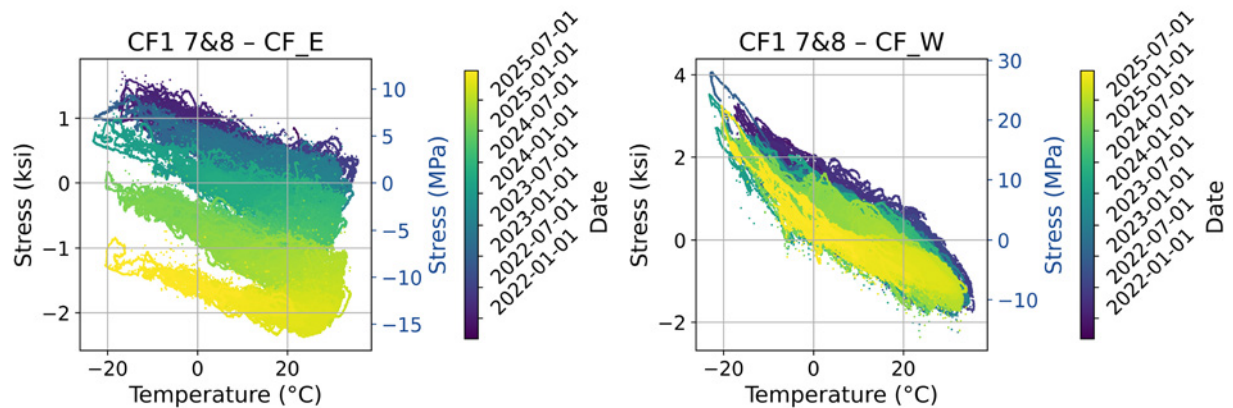


Figure 183. Graph. Temperature vs. Stress Data collected from CF1 7&8.

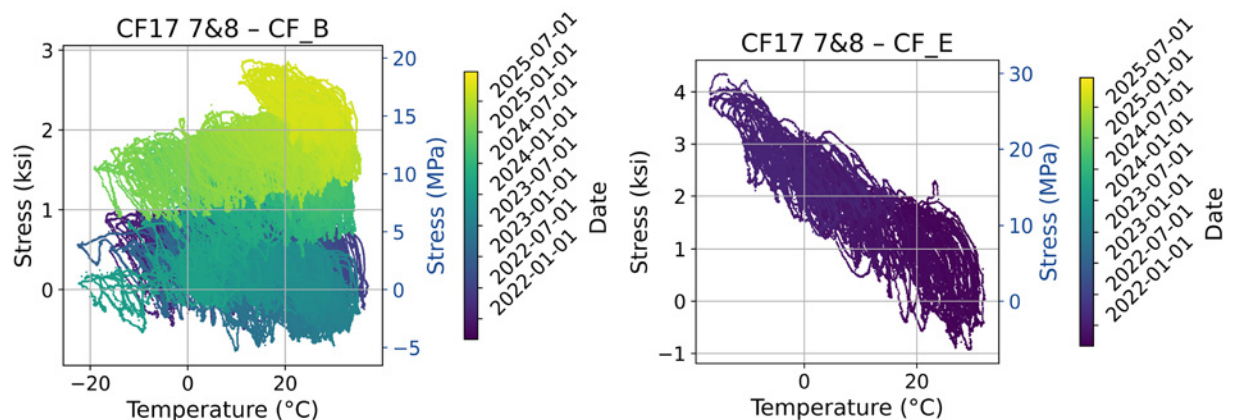


Figure 184. Graph. Temperature vs. stress data collected from CF17 7&8.

Crackmeters

A total of 4 crackmeters (denoted as “C” in Figure 2) were installed on Mattis-57 bridge. These were installed to measure relative displacement between the approach slab and transition slab (at the deck level), in the longitudinal direction (parallel to the bridge longitudinal axis). The installation process and exact locations are described in detail in Volume I (Zhou et al. 2026b).

To provide an overview of the recorded response, Table 30 summarizes the key measures for all crackmeters, including maximum, minimum, and overall range of displacement values over the monitoring period.

Table 30. Summary of Crackmeter Displacement Data—Mattis-57

Orientation	Location	Corner Type	Min. Displ. cm (in.)	Temp. at Min. (°C)	Max. Displ. cm (in.)	Temp. at Max. (°C)	Displ. Range cm (in.)
Par.	NE	acute	-0.61 (-0.24)	35.6	2.67 (1.05)	-18.0	3.28 (1.29)
Par.	NW	obtuse	-0.79 (-0.31)	31.0	1.95 (0.77)	-16.8	2.74 (1.08)
Par.	SE	obtuse	-0.86 (-0.34)	36.9	1.92 (0.76)	-12.7	2.78 (1.09)
Par.	SW	acute	-0.85 (-0.34)	33.1	2.51 (0.99)	-18.1	3.36 (1.32)

All data corresponding to the summary presented in Table 30 is illustrated in the response history plots below (Figure 185 and Figure 186), which show the full temporal evolution of the recorded displacements.

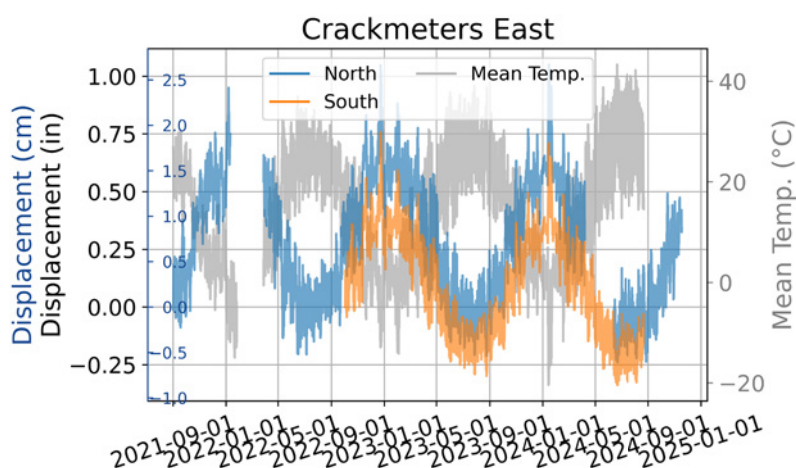


Figure 185. Graph. Displacement data collected from east crackmeters (parallel).

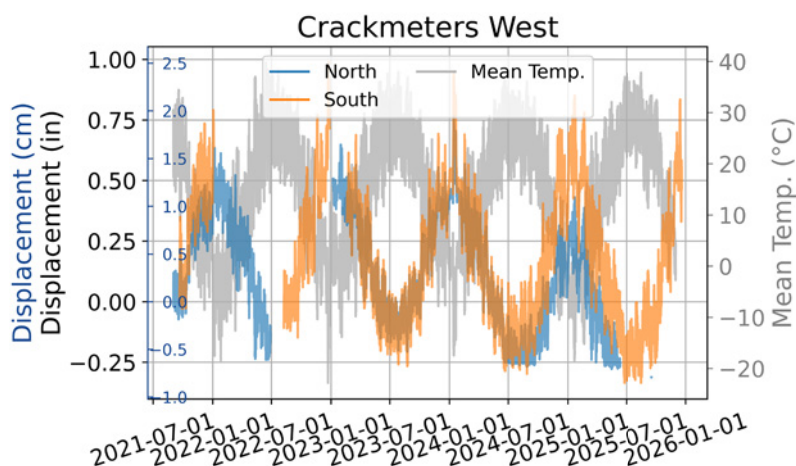


Figure 186. Graph. Displacement data collected from west crackmeters (parallel).

The same displacement data presented above, is presented in temperature versus displacement readings in the plots below (Figure 187 and Figure 188). The temperature used for these plots is the average temperature collected from thermistors attached to all crackmeters on each plot.

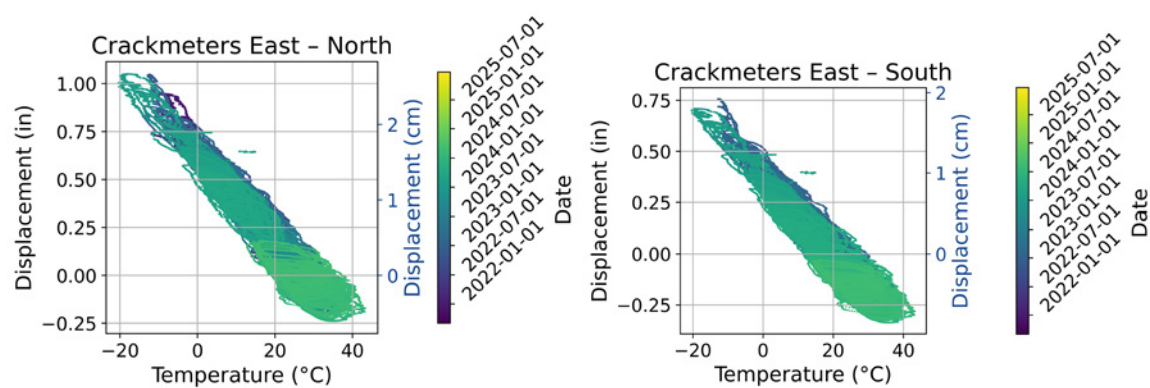


Figure 187. Graph. Temperature vs. displacement data collected from east crackmeters (parallel).

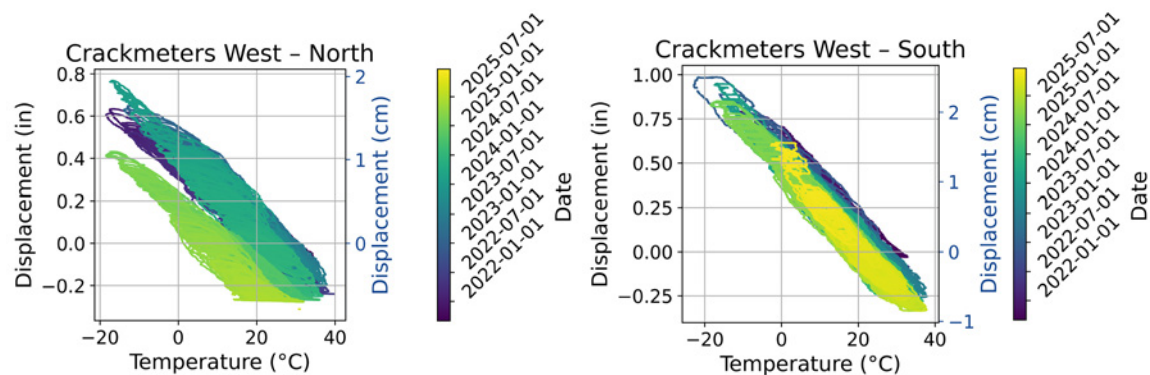


Figure 188. Graph. Temperature vs. displacement data collected from west crackmeters (parallel).

Tiltmeters

A total of 2 tiltmeters (denoted as “T” in Figure 2) were installed on Mattis-57 bridge. These were installed to measure rotation at the bridge ends, and were mounted at mid-height on the abutment backwall near girder 6 (G6 in Figure 2).

To provide an overview of the recorded response, Table 31 summarizes the key measures for both tiltmeters, including maximum, minimum, and overall range of rotation values over the monitoring period.

Table 31. Summary of Rotation Measurements—Mattis-57

Location	Min. Rotation (deg)	Temp. at Min. (°C)	Max. Rotation (deg)	Temp. at Max. (°C)	Rotation Range (deg)
North	-0.05	0.1	0.17	12.7	0.22
South	-0.27	22.3	0.13	2.4	0.40

All data corresponding to the summary presented in Table 31 is illustrated in Figure 189, which shows the full temporal evolution of the recorded rotations.

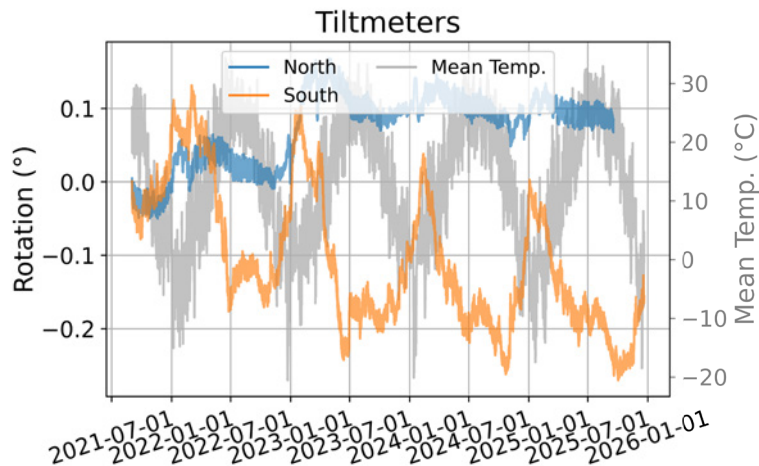


Figure 189. Graph. Rotation data collected from tiltmeters.

The same rotation data presented above, is presented in temperature versus rotation form in Figure 190. The temperature used for these plots is the average temperature collected from thermistors attached to all tiltmeters on each plot.

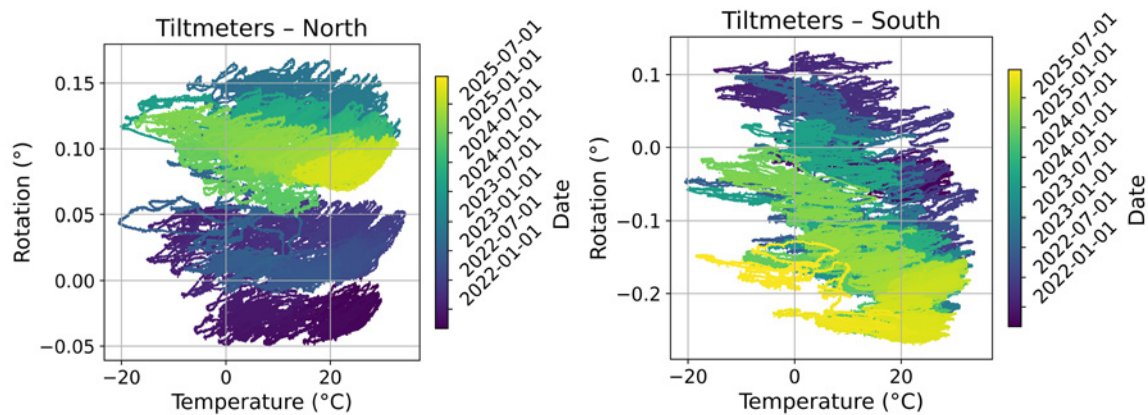


Figure 190. Graph. Temperature vs. rotation data collected from tiltmeters.

APPENDIX B: VALIDATION OF INTEGRAL ABUTMENT BRIDGE MODEL UNDER THERMAL LOADS

This appendix presents the complete set of validation results for the integral abutment bridge (IAB) model subjected to thermal loading. While the main body of the report focuses on representative cross-sections to highlight key behavioral trends, this appendix provides the full validation dataset for all instrumented gauges included in the monitoring program. For each gauge location, comparisons between FE-predicted and field-measured responses are presented using temperature-stress relationships, as well as temperature-decomposed stress relationships. Additionally, response history comparisons between FE predictions and field measurements are presented for the raw stress data at all cross-sections. These results provide a comprehensive assessment of the ability of the model to reproduce thermally induced behavior across the full range of instrumented locations, complementing the representative results discussed in the main body of the report.

STRESS DATA FROM GIRDER CROSS-SECTIONS—VALIDATION

Figures 191 through 203 present the comparison between FE-predicted and field-measured stresses as a function of temperature for all instrumented gauge locations of the monitored IAB. These plots illustrate the direct relationship between thermal loading and the resulting structural response, enabling an assessment of the ability of the model to capture the magnitude and trend of thermally induced stresses across the bridge.

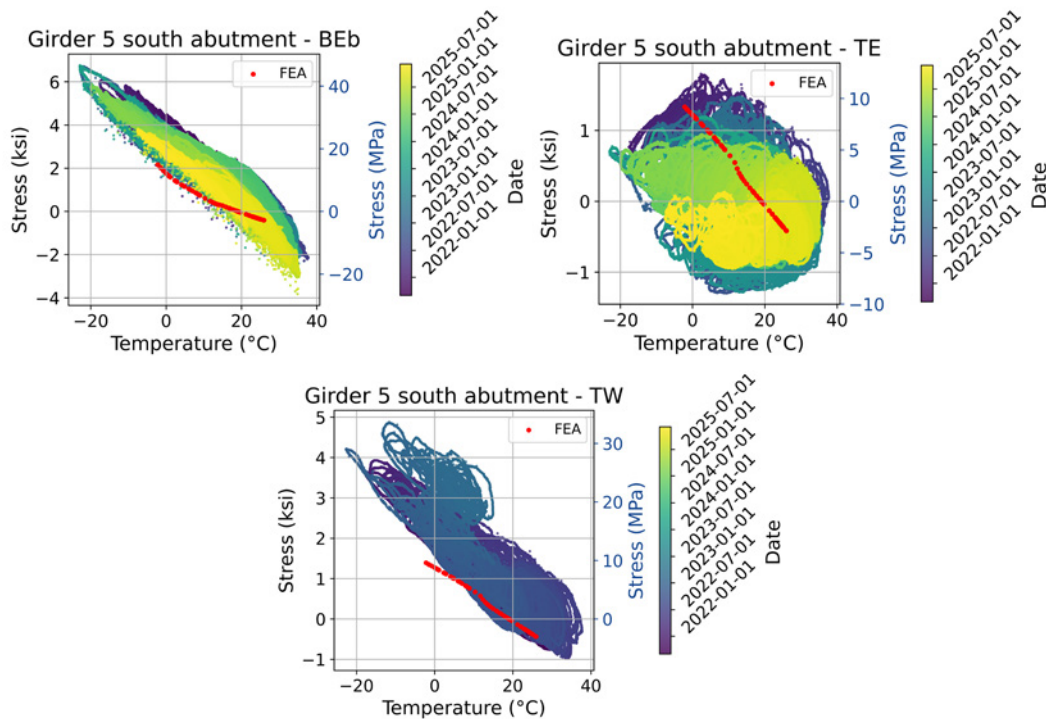


Figure 191. Graph. Temperature-stress comparison (FE vs. field data) at Girder 5 South Abutment.

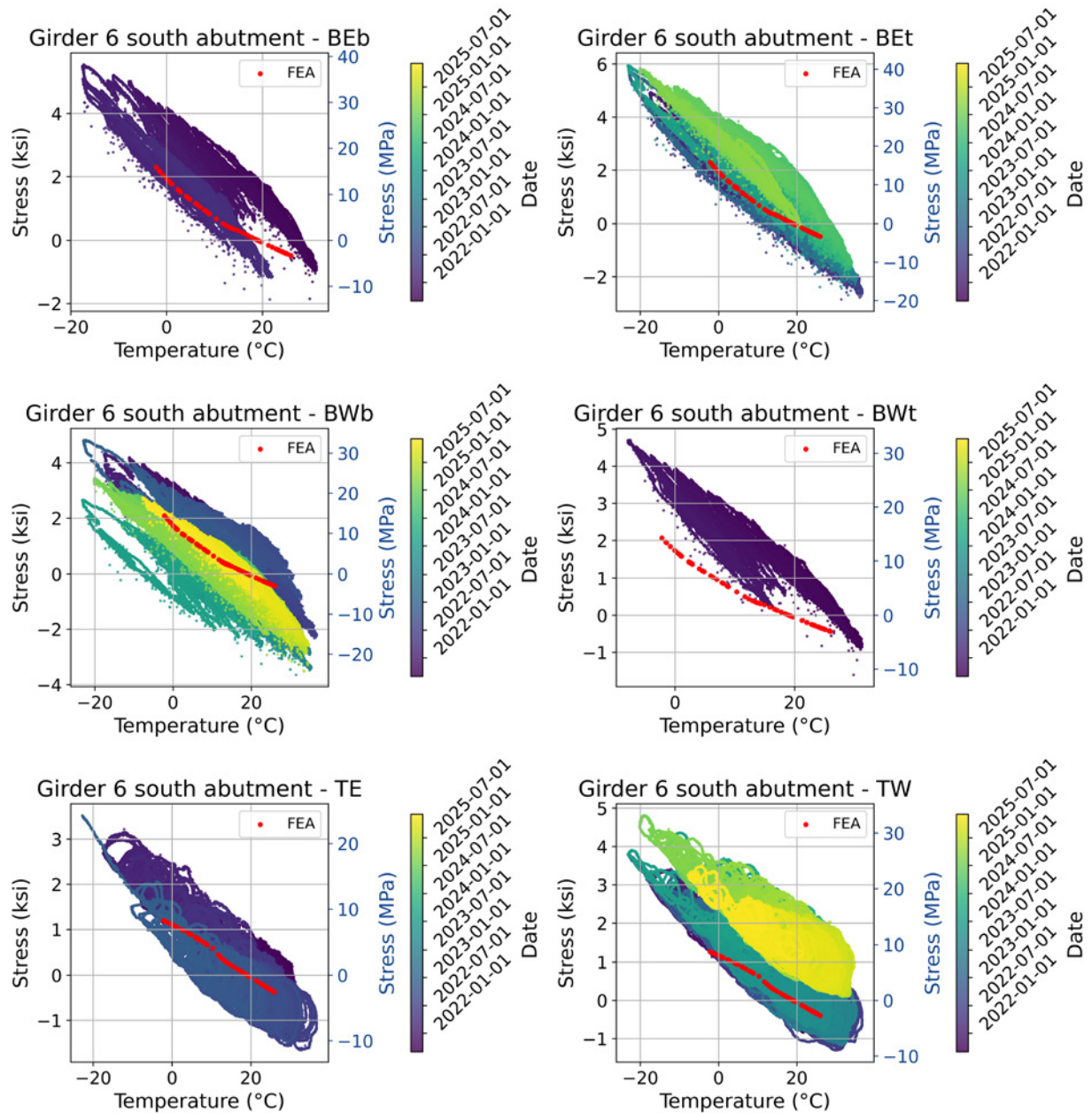


Figure 192. Graph. Temperature-stress comparison (FE vs. field data) at Girder 6 South Abutment.

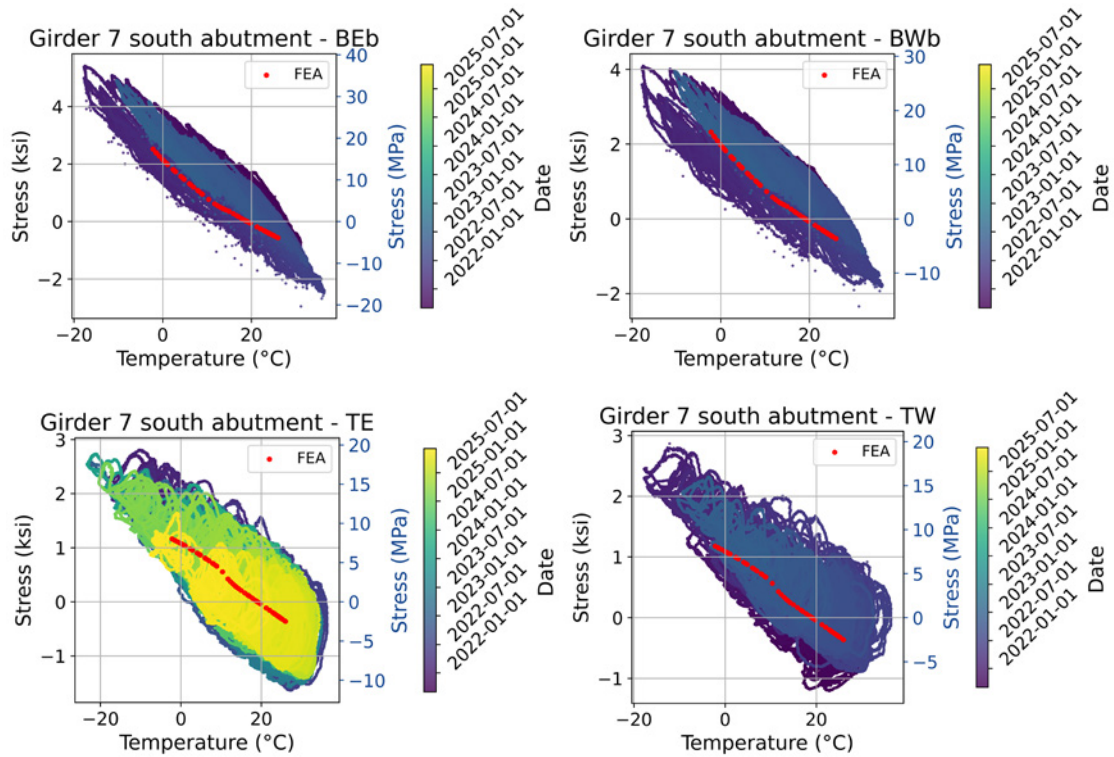


Figure 193. Graph. Temperature-stress comparison (FE vs. field data) at Girder 7 South Abutment.

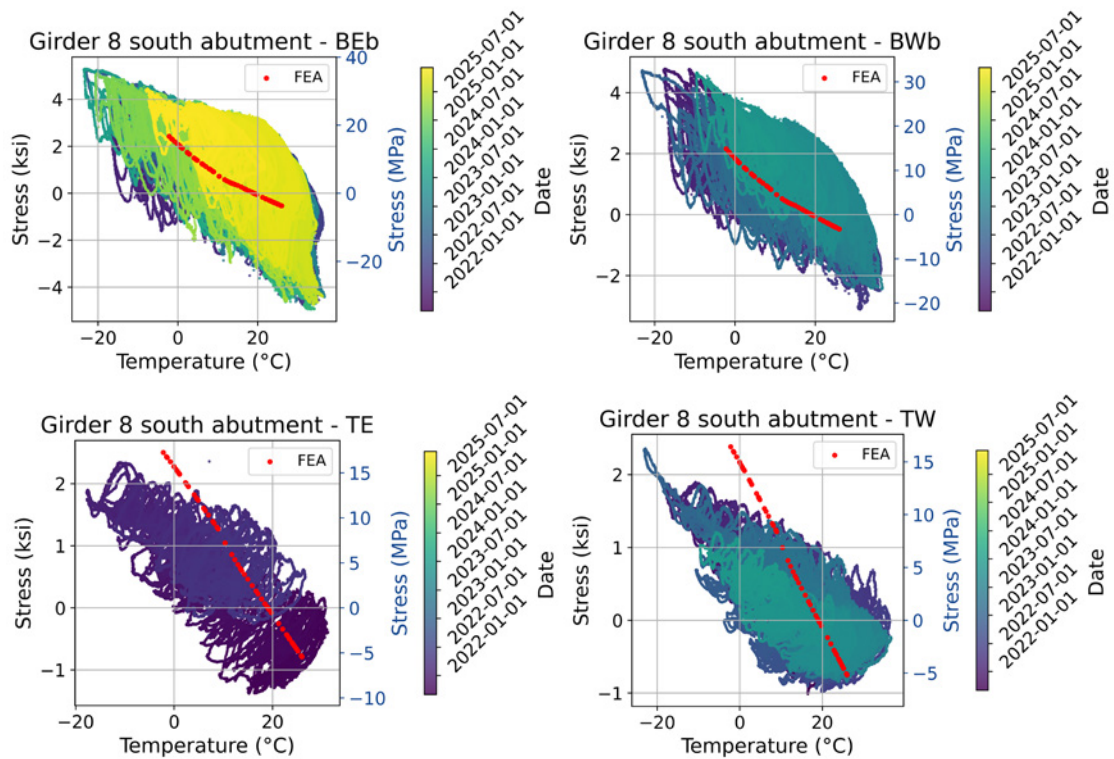


Figure 194. Graph. Temperature-stress comparison (FE vs. field data) at Girder 8 South Abutment.

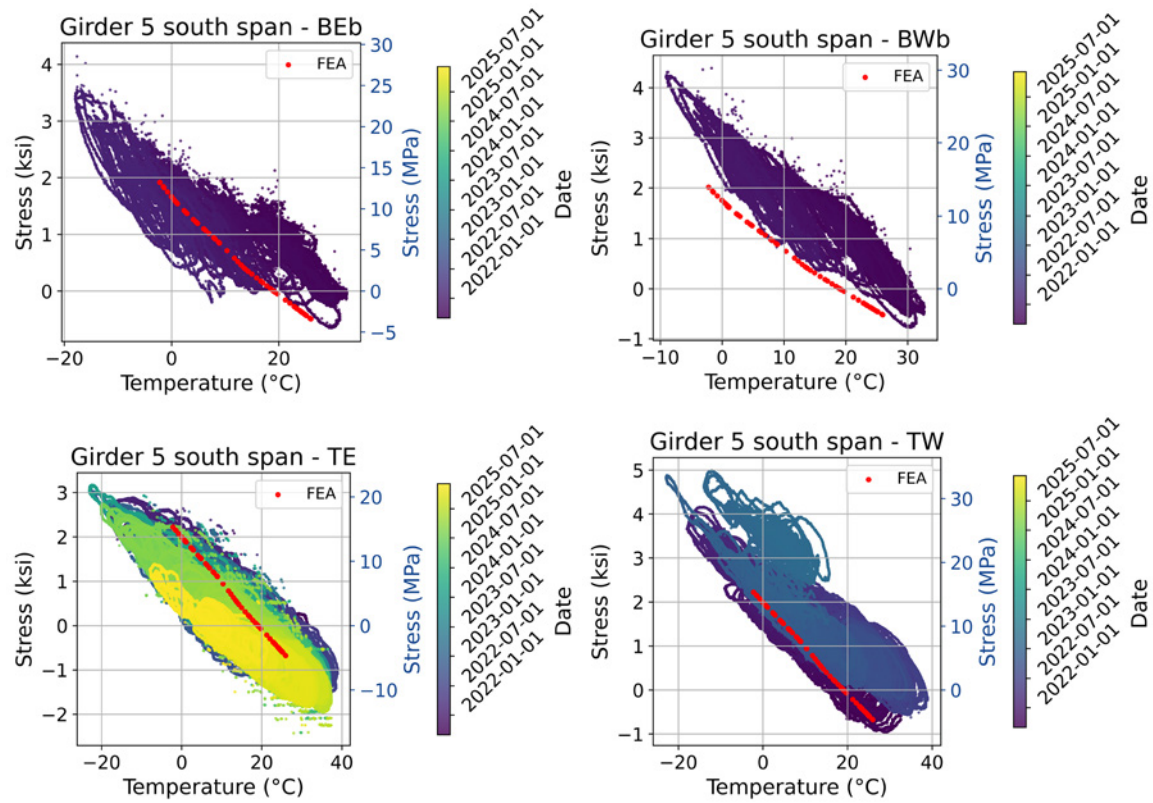


Figure 195. Graph. Temperature-stress comparison (FE vs. field data) at Girder 5 South Span.

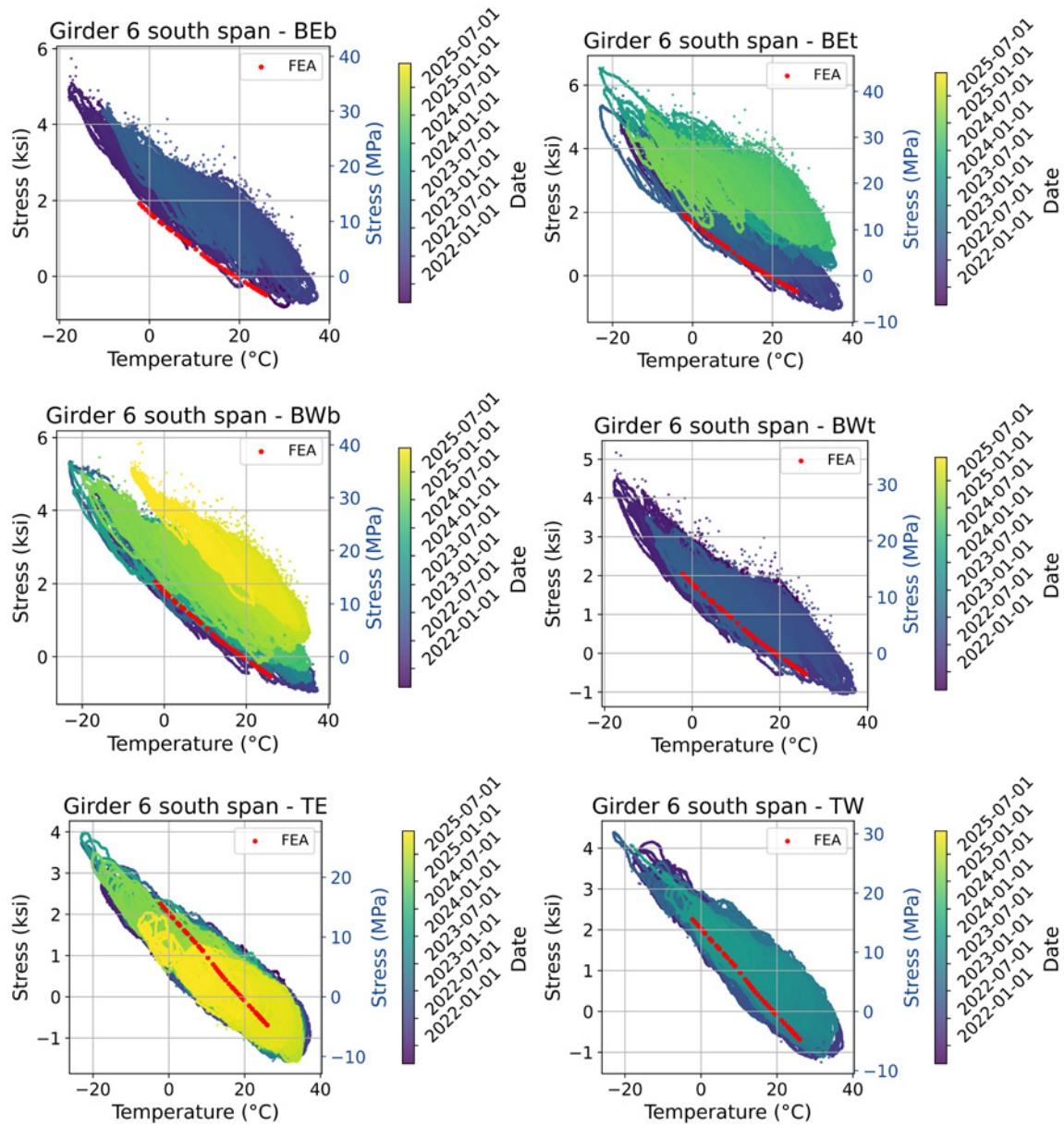


Figure 196. Graph. Temperature-stress comparison (FE vs. field data) at Girder 6 South Span.

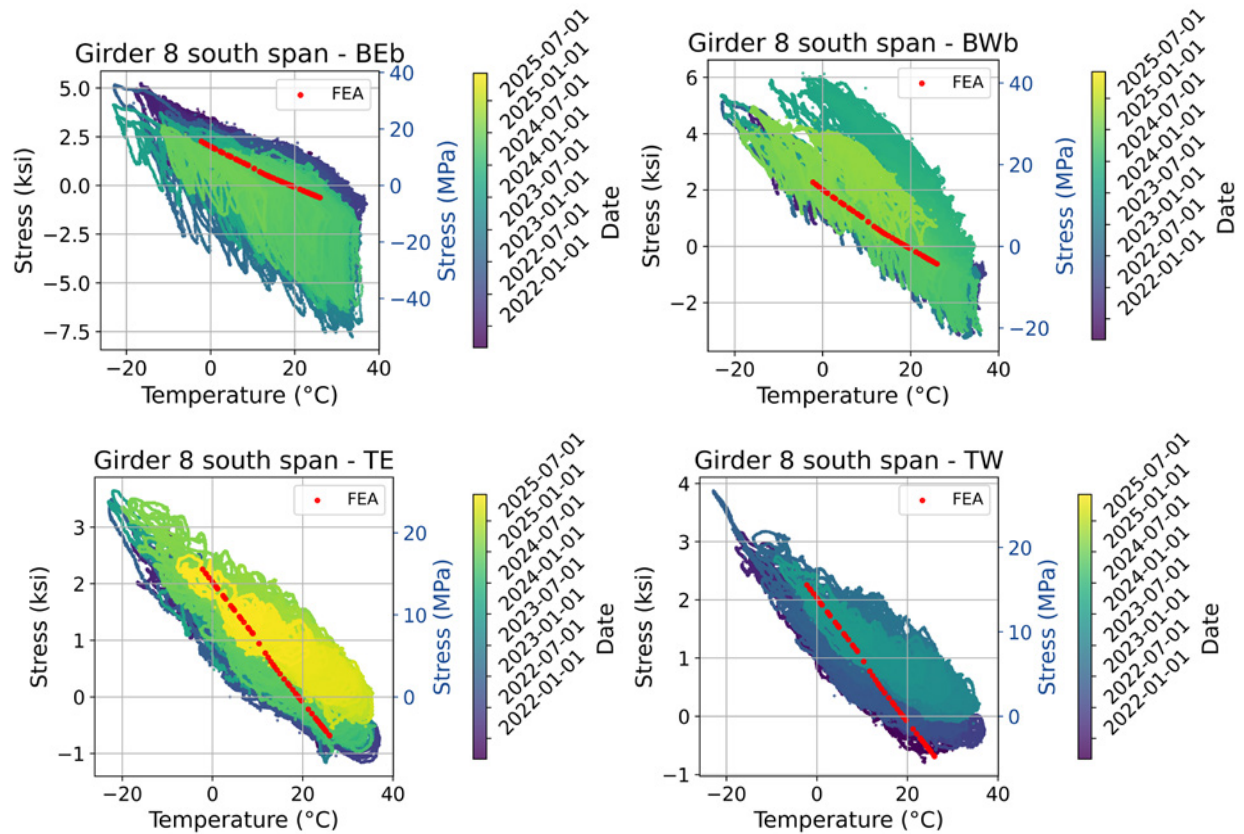


Figure 197. Graph. Temperature-stress comparison (FE vs. field data) at Girder 8 South Span.

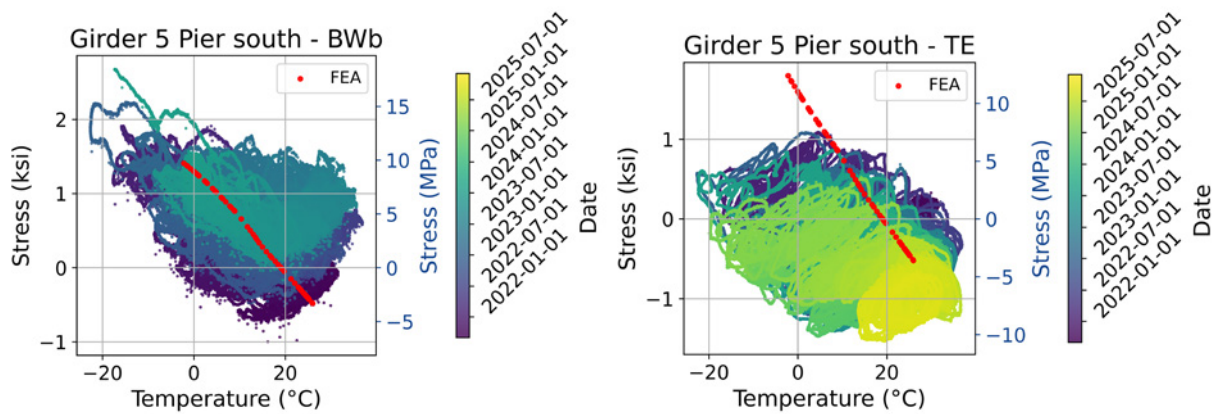


Figure 198. Graph. Temperature-stress comparison (FE vs. field data) at Girder 5 Pier South.

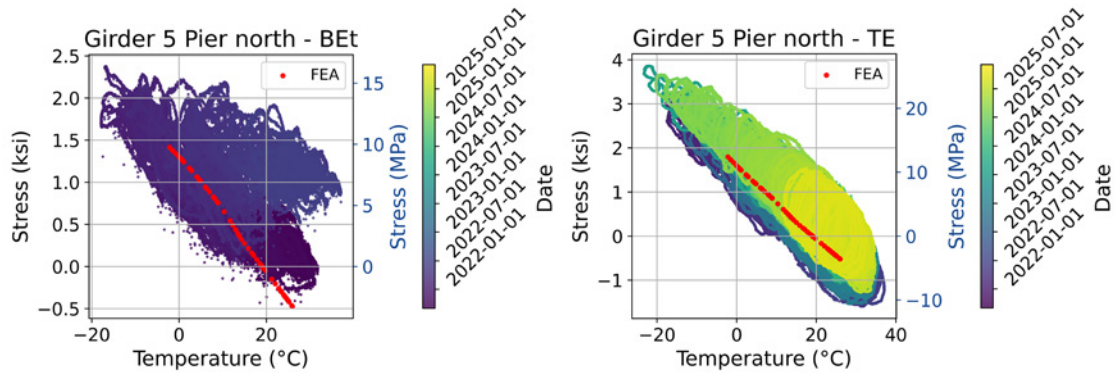


Figure 199. Graph. Temperature-stress comparison (FE vs. field data) at Girder 5 Pier North.

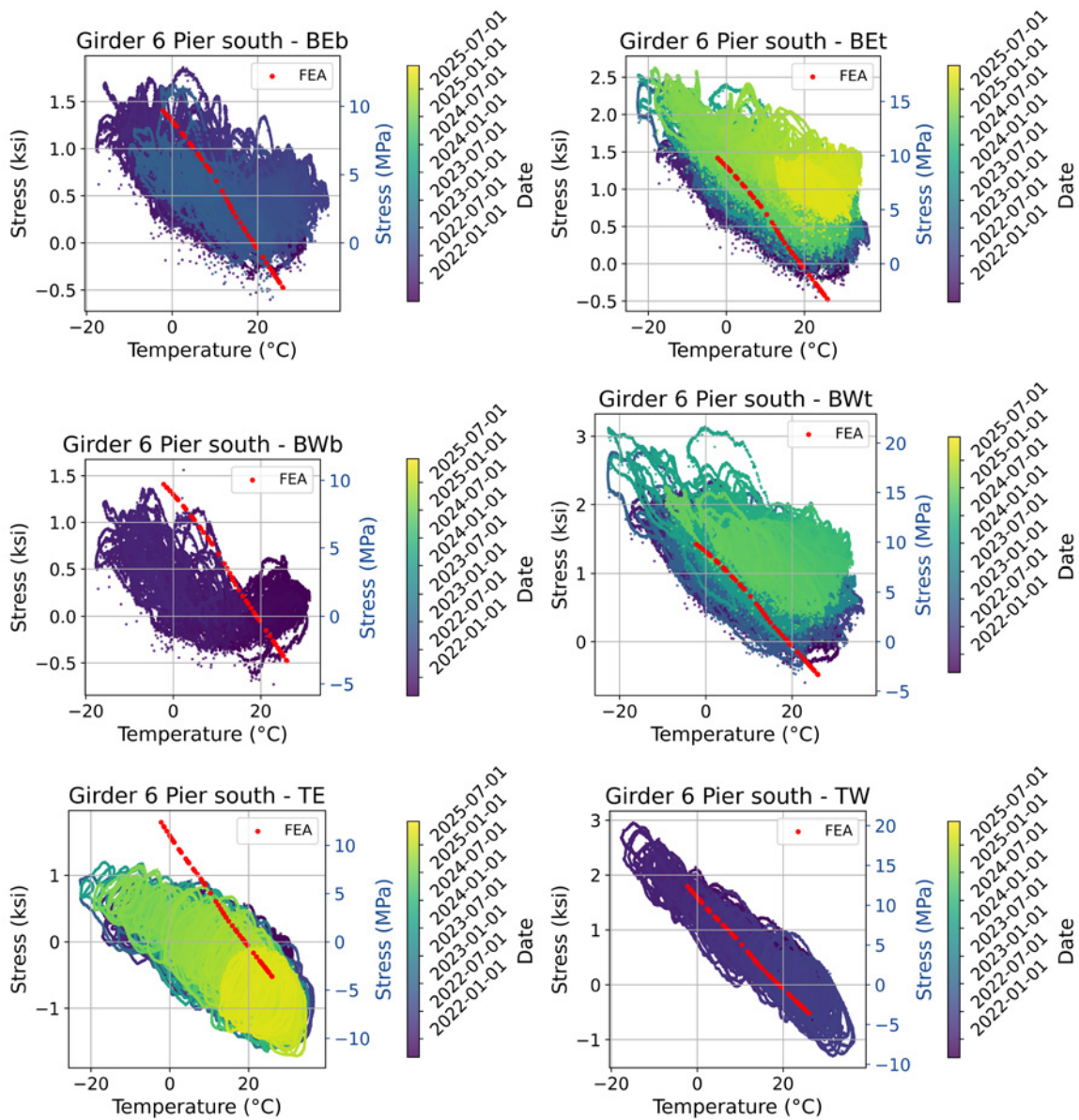


Figure 200. Graph. Temperature-stress comparison (FE vs. field data) at Girder 6 Pier South.

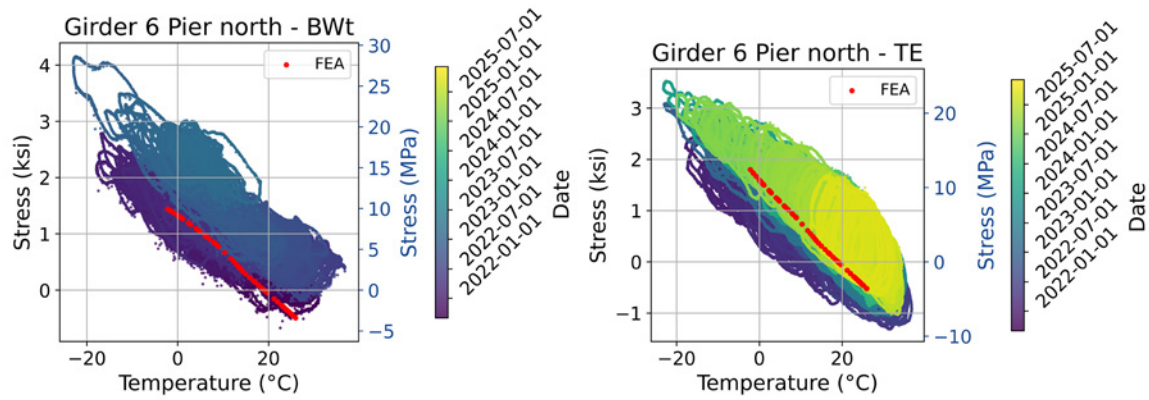


Figure 201. Graph. Temperature-stress comparison (FE vs. field data) at Girder 6 Pier North.

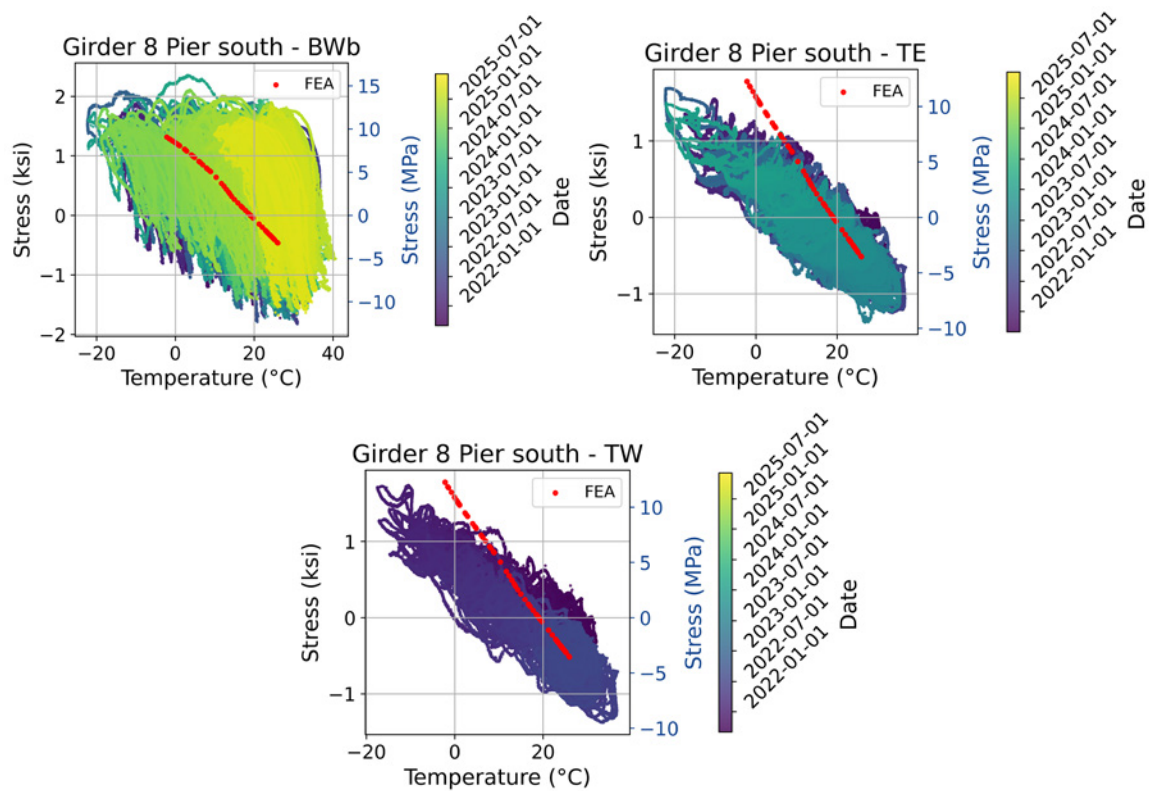


Figure 202. Graph. Temperature-stress comparison (FE vs. field data) at Girder 8 Pier.

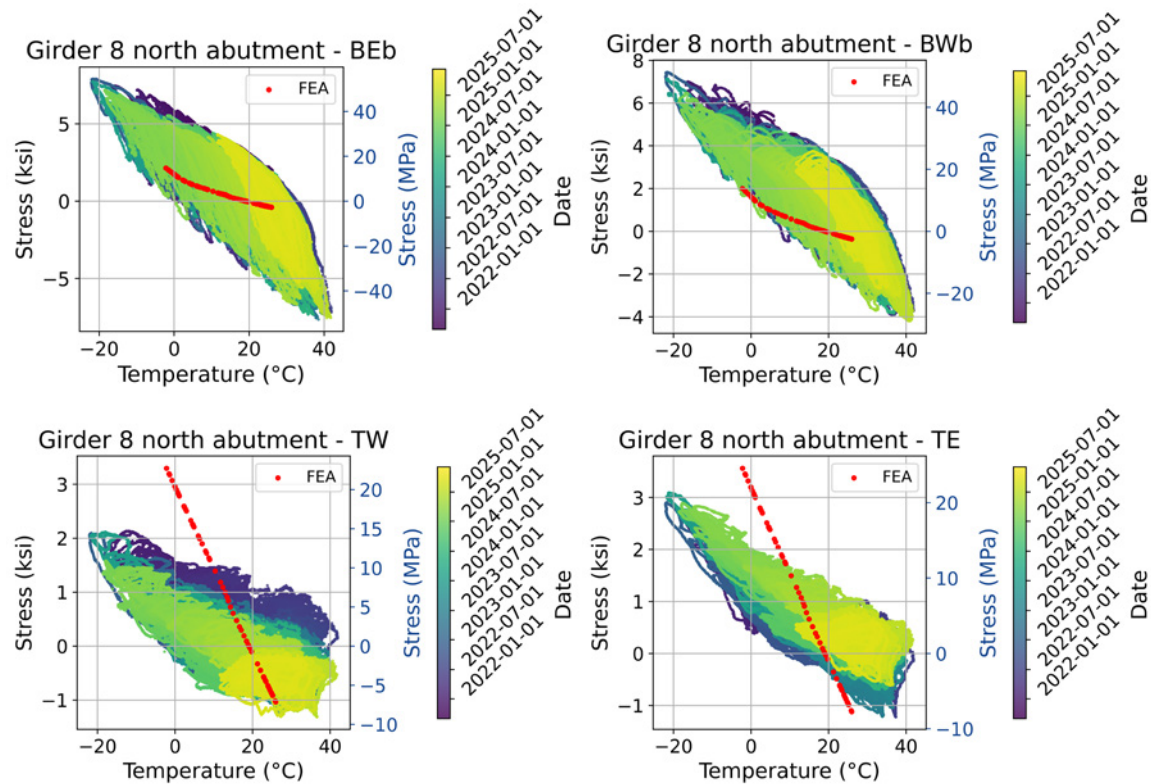


Figure 203. Graph. Temperature-stress comparison (FE vs. field data) at Girder 8 North Abutment.

Additionally, response history plots shown in Figures 204 through 216 compare the field-measured stresses with the FE results over the entire monitoring period. Emphasis is placed on bottom flange gauge readings. In these figures, the raw field temperature, the processed temperature input used in the FE model (i.e., a downsampled moving mean as discussed in the main body of the document), the FE-extracted stresses, and the measured field stresses are presented concurrently. To ensure consistency in the comparison, a moving mean of the field-measured stresses—computed using the same window employed for the temperature processing—was also included in the response history plots, enabling a direct evaluation of long-term trends between the numerical and field response.

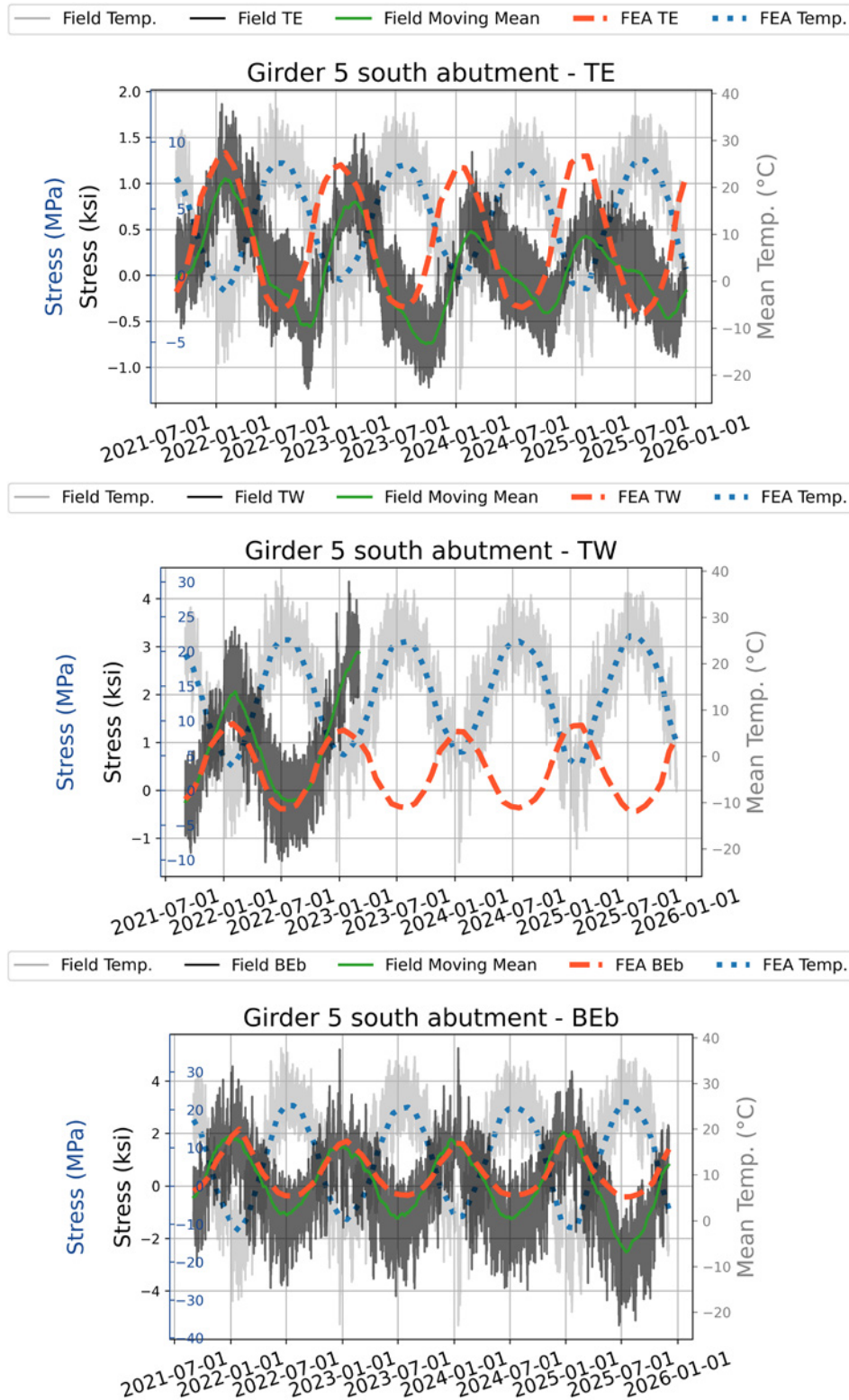
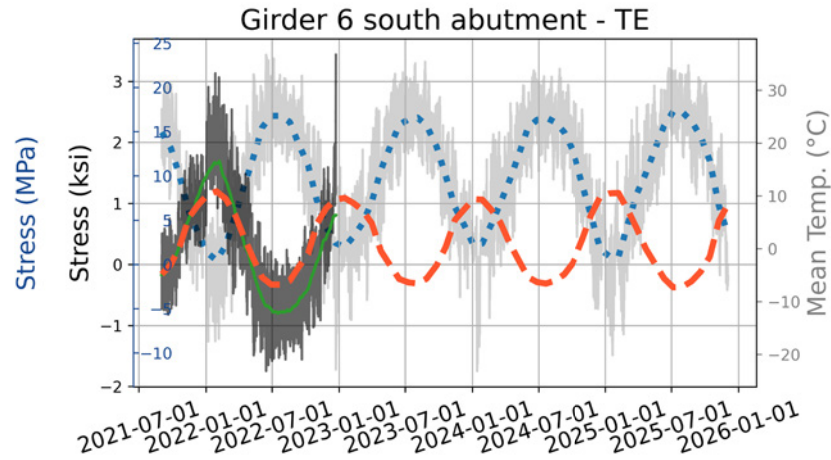
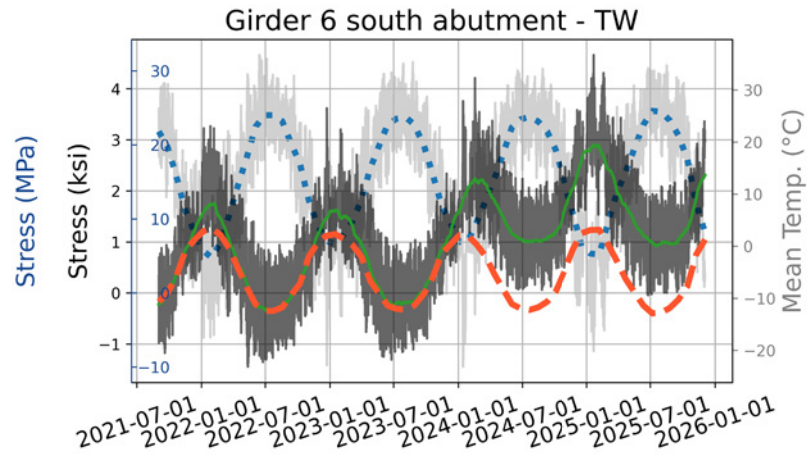


Figure 204. Graph. Response history stress comparison (FE vs. field data) at Girder 5 South Abutment.

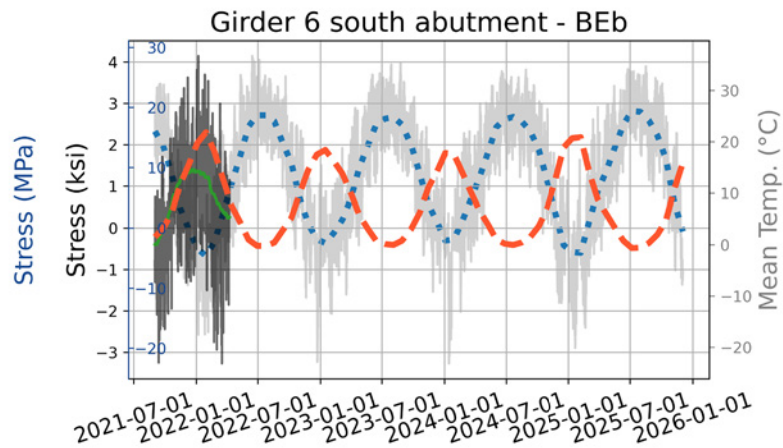
— Field Temp. — Field TE — Field Moving Mean - - FEA TE . . . FEA Temp.



— Field Temp. — Field TW — Field Moving Mean - - FEA TW . . . FEA Temp.



— Field Temp. — Field BEb — Field Moving Mean - - FEA BEb . . . FEA Temp.



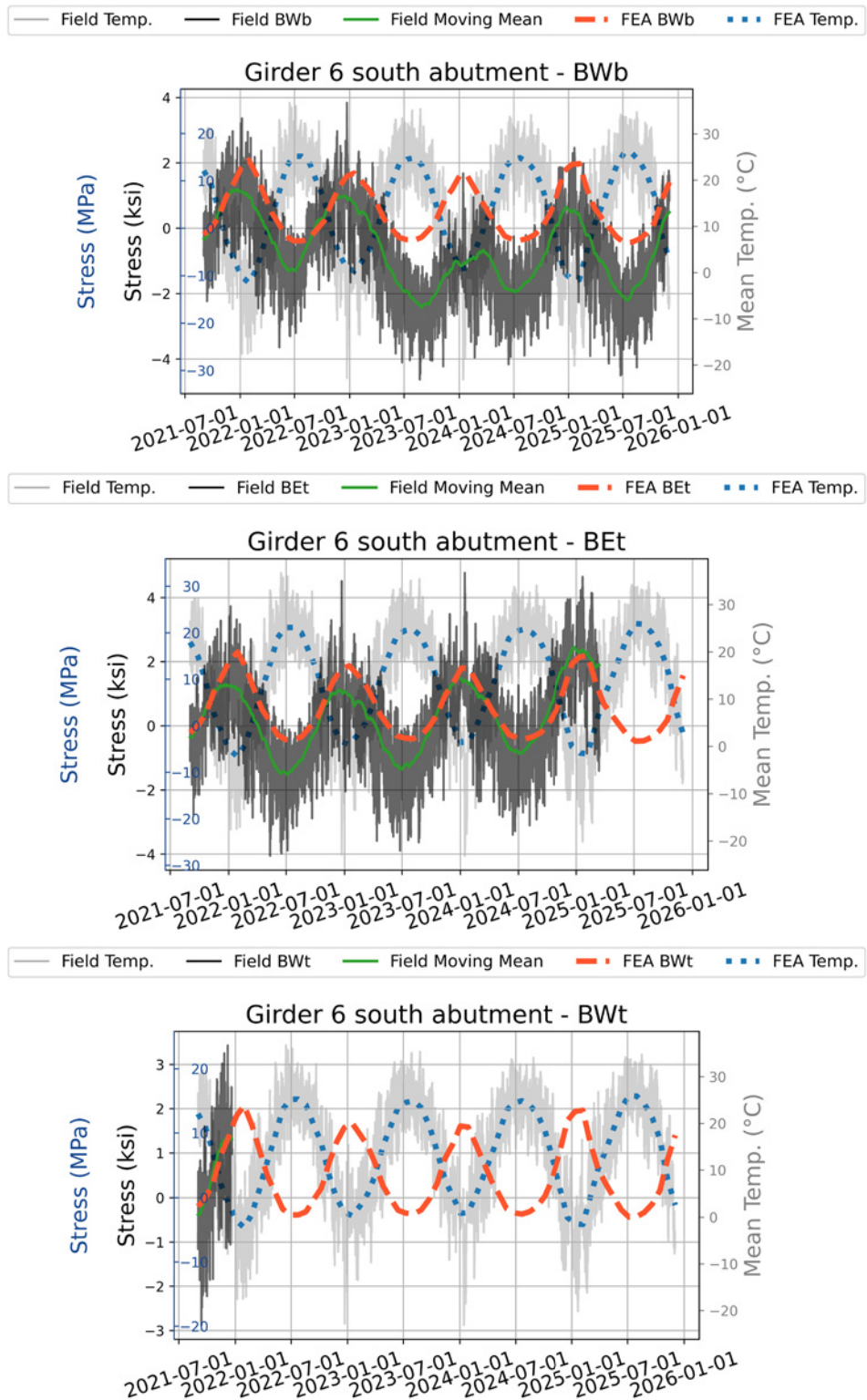
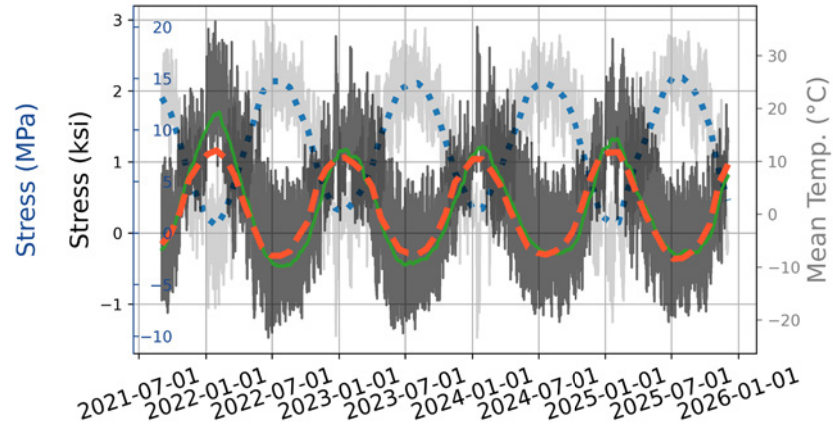


Figure 205. Graph. Response history stress comparison (FE vs. field data) at Girder 6 South Abutment.

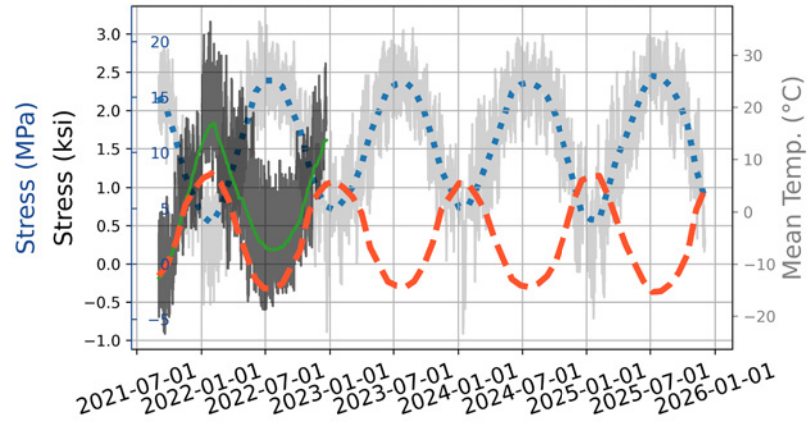
— Field Temp. — Field TE — Field Moving Mean — FEA TE ■ FEA Temp.

Girder 7 south abutment - TE



— Field Temp. — Field TW — Field Moving Mean — FEA TW ■ FEA Temp.

Girder 7 south abutment - TW



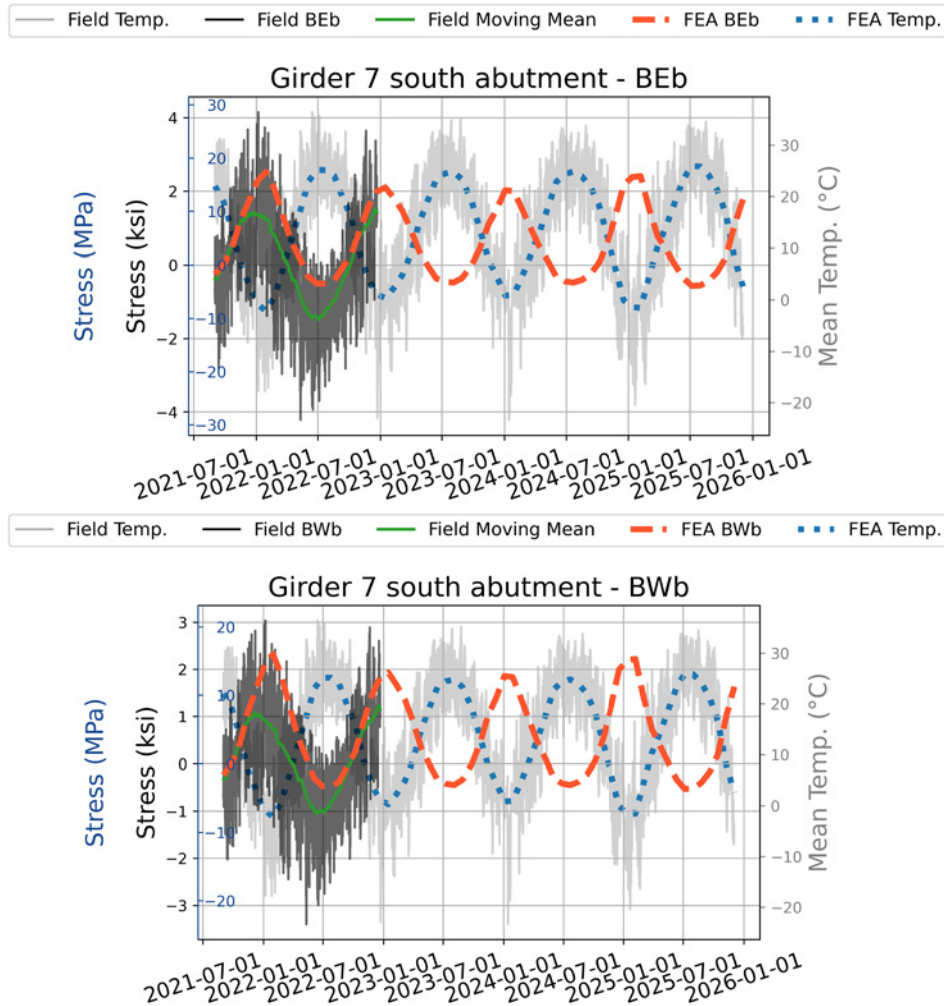
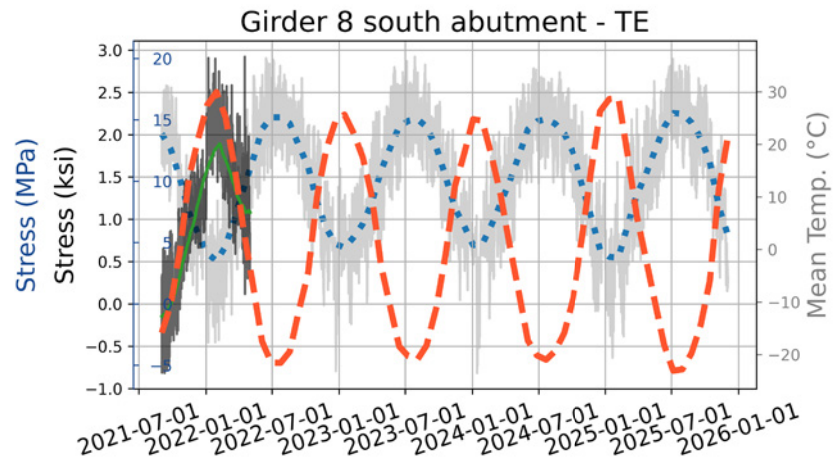
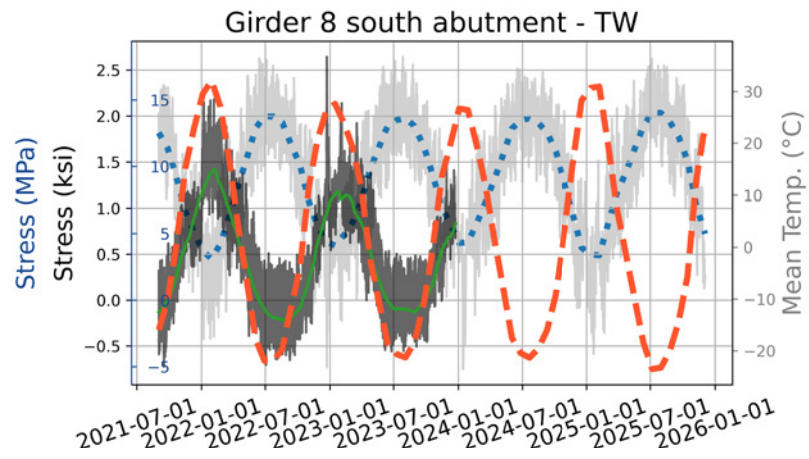


Figure 206. Graph. Response history stress comparison (FE vs. field data) at Girder 7 South Abutment.

— Field Temp. — Field TE — Field Moving Mean - - FEA TE . . FEA Temp.



— Field Temp. — Field TW — Field Moving Mean - - FEA TW . . FEA Temp.



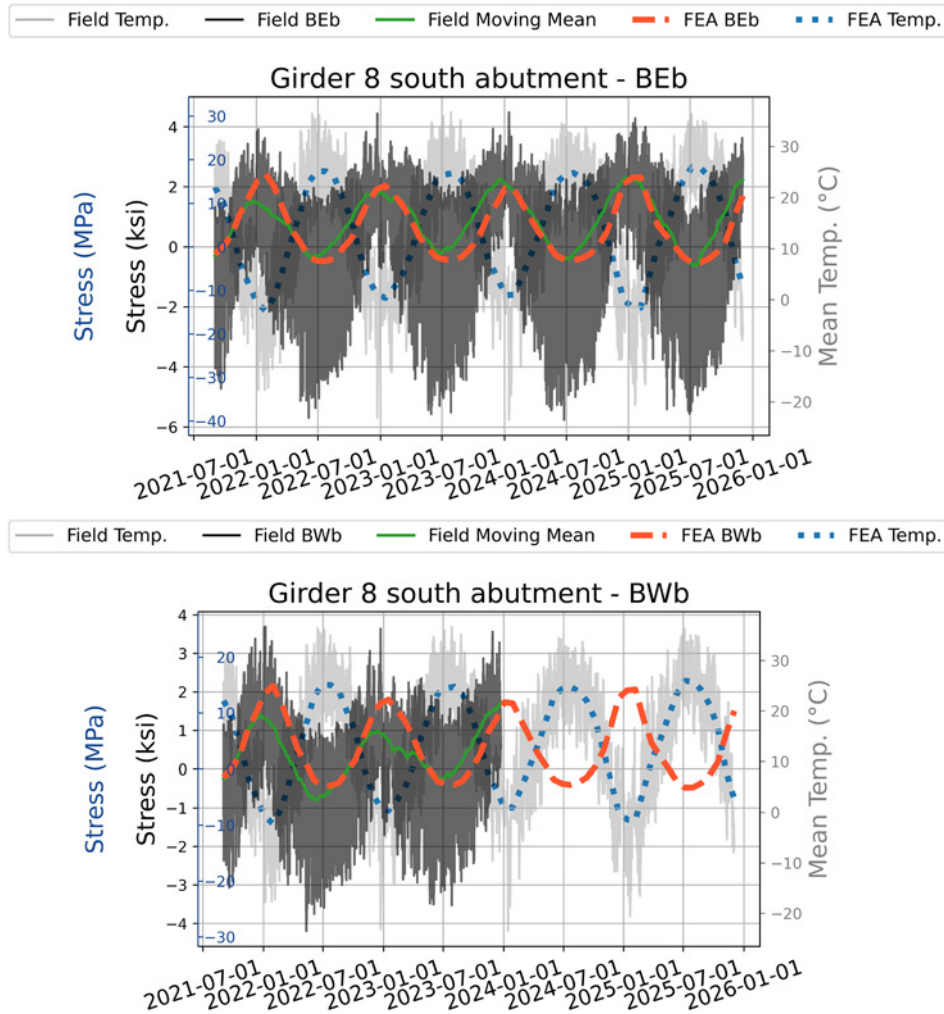
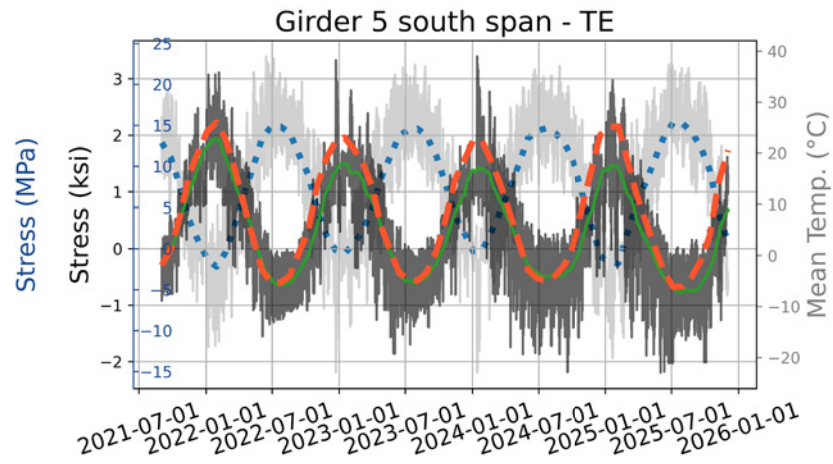
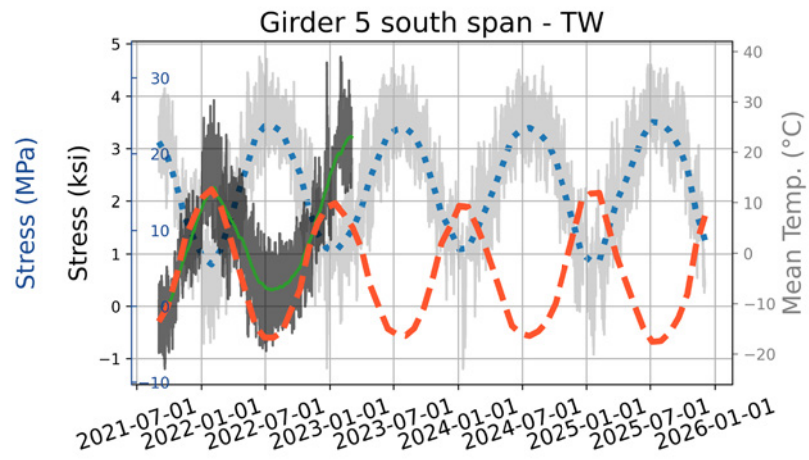


Figure 207. Graph. Response history stress comparison (FE vs. field data) at Girder 8 South Abutment.

— Field Temp. — Field TE — Field Moving Mean — FEA TE ■ FEA Temp.



— Field Temp. — Field TW — Field Moving Mean — FEA TW ■ FEA Temp.



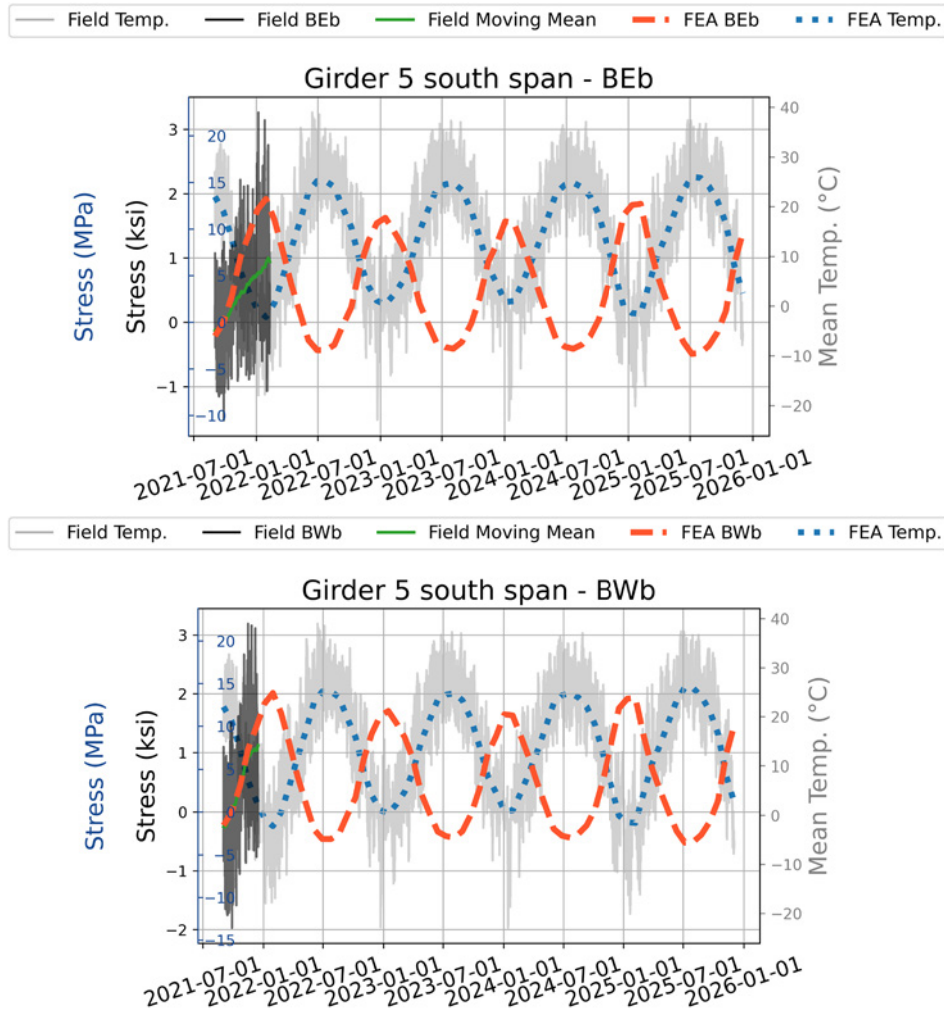
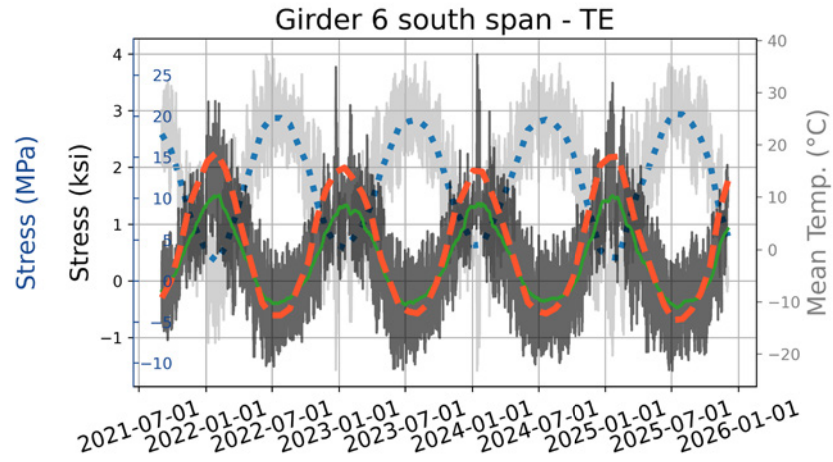
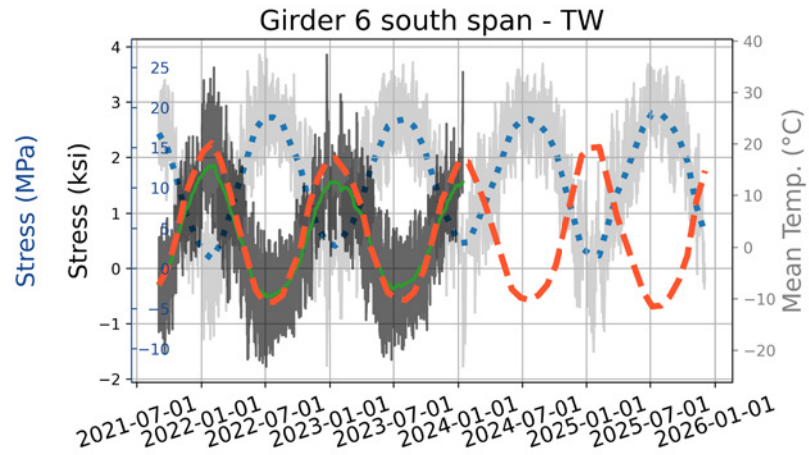


Figure 208. Graph. Response history stress comparison (FE vs. field data) at Girder 5 South Span.

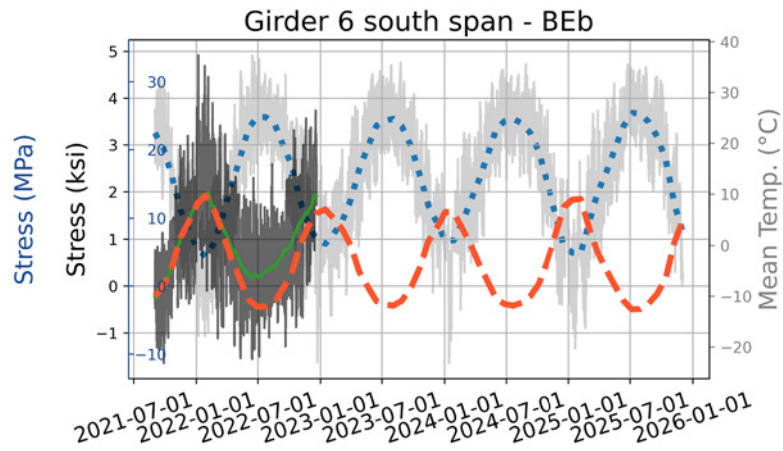
— Field Temp. — Field TE — Field Moving Mean — FEA TE ■ FEA Temp.



— Field Temp. — Field TW — Field Moving Mean — FEA TW ■ FEA Temp.



— Field Temp. — Field BEb — Field Moving Mean — FEA BEb ■ FEA Temp.



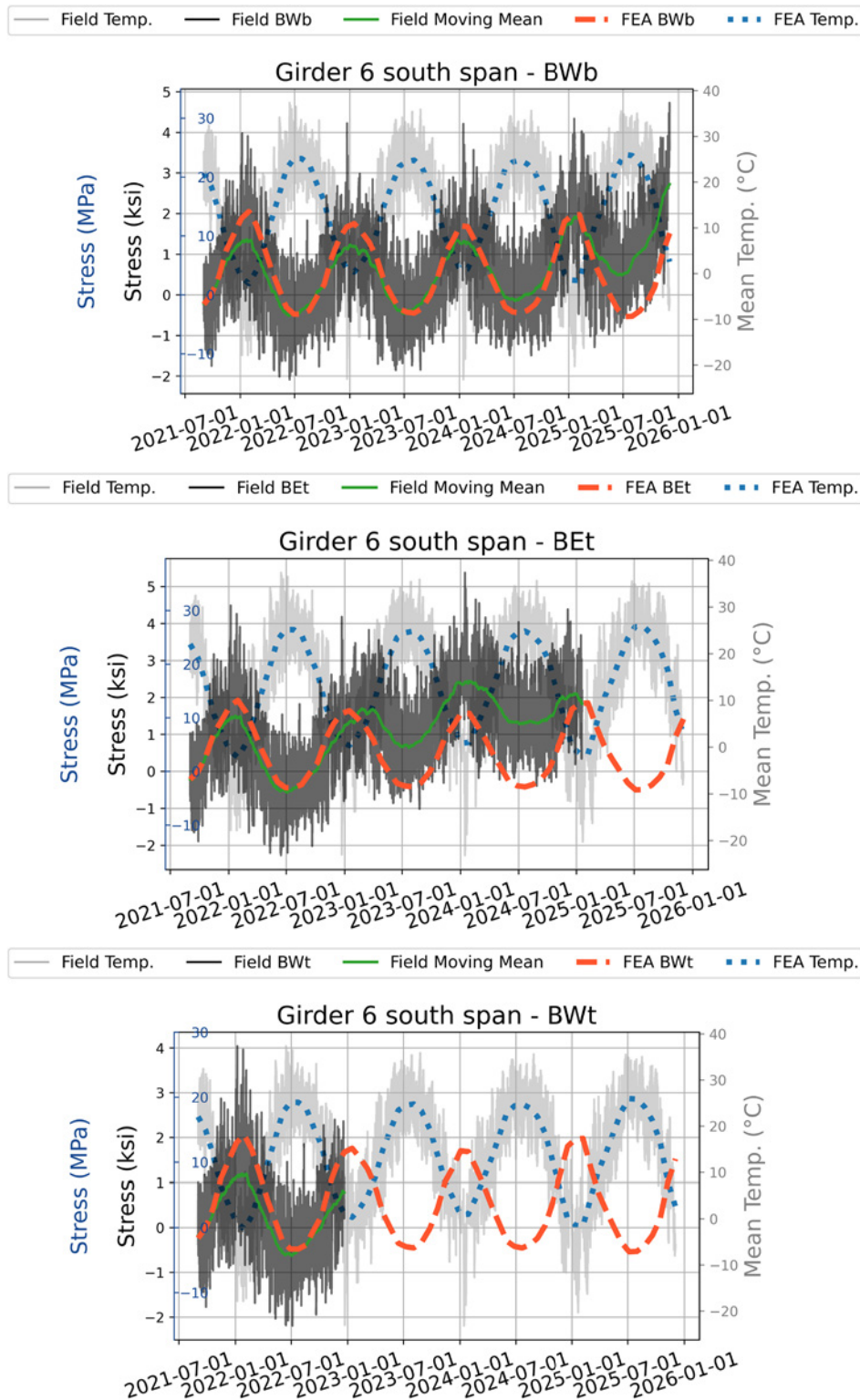
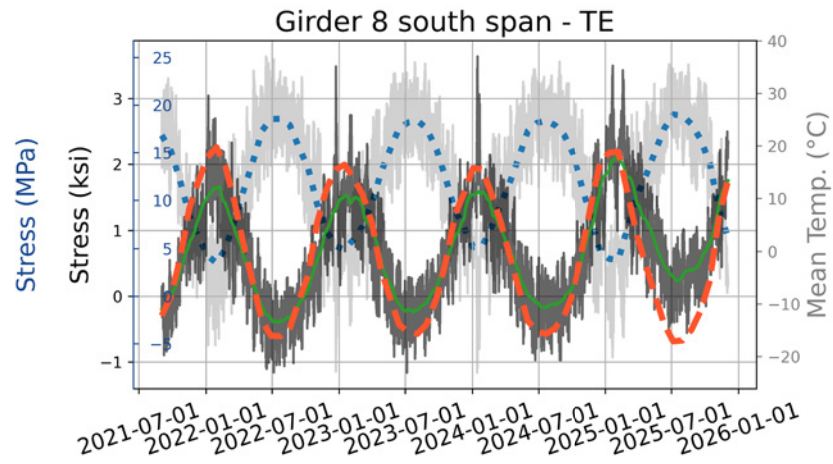
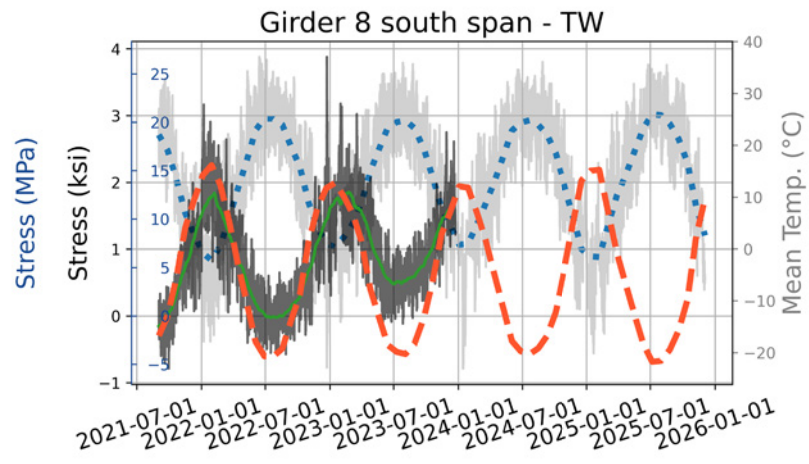


Figure 209. Graph. Response history stress comparison (FE vs. field data) at Girder 6 South Span.

— Field Temp. — Field TE — Field Moving Mean - - FEA TE ··· FEA Temp.



— Field Temp. — Field TW — Field Moving Mean - - FEA TW ··· FEA Temp.



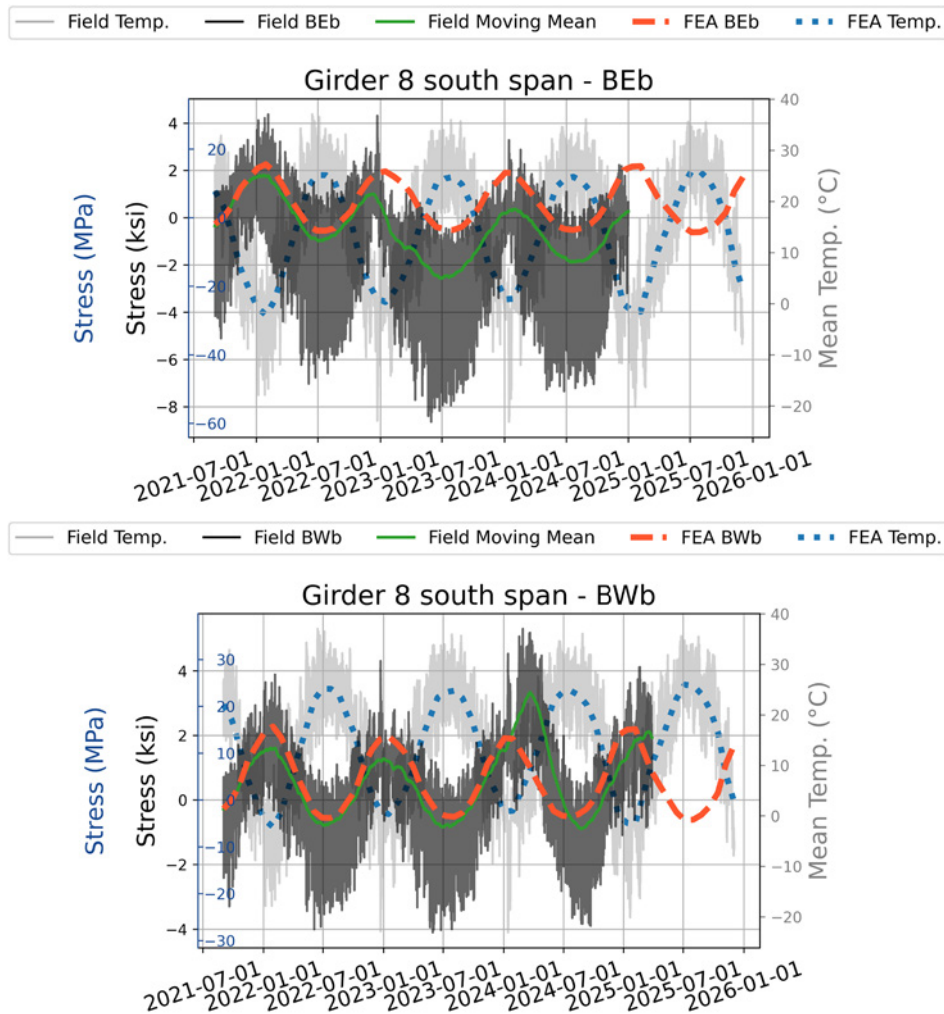


Figure 210. Graph. Response history stress comparison (FE vs. field data) at Girder 8 South Span.

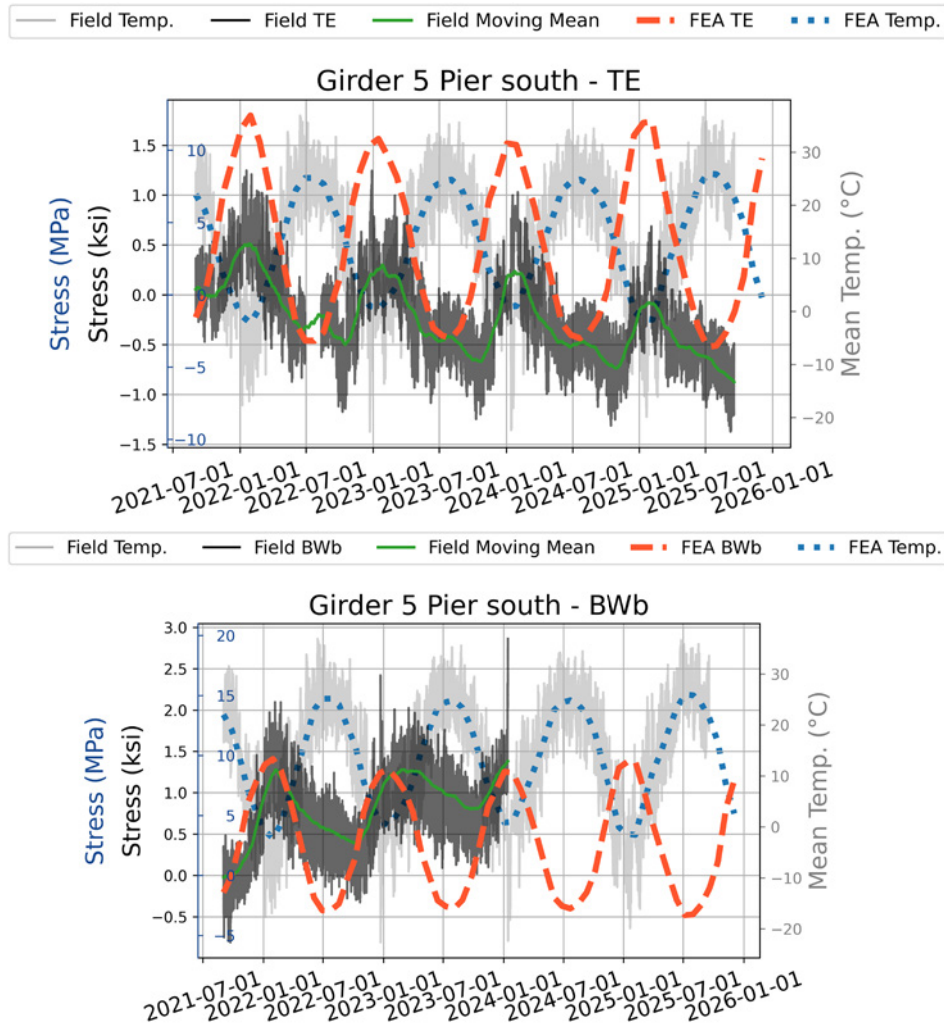


Figure 211. Graph. Response history stress comparison (FE vs. field data) at Girder 5 Pier South.

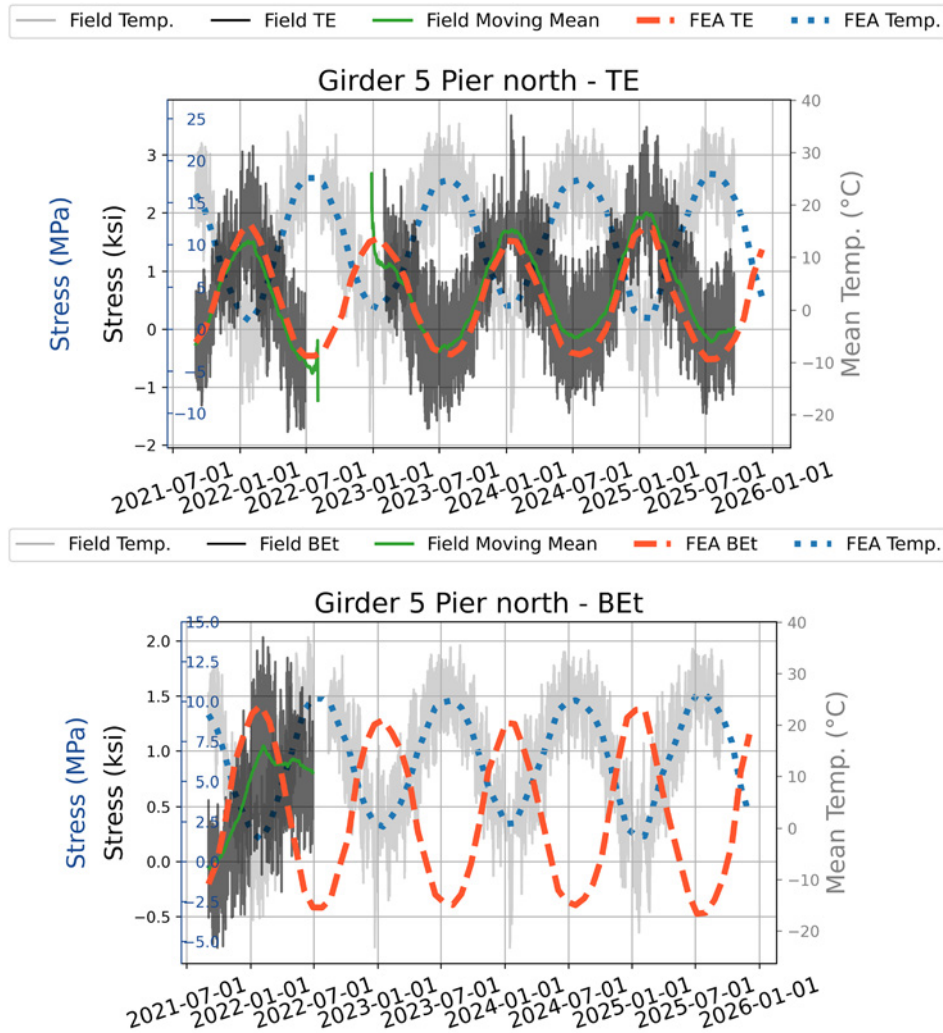
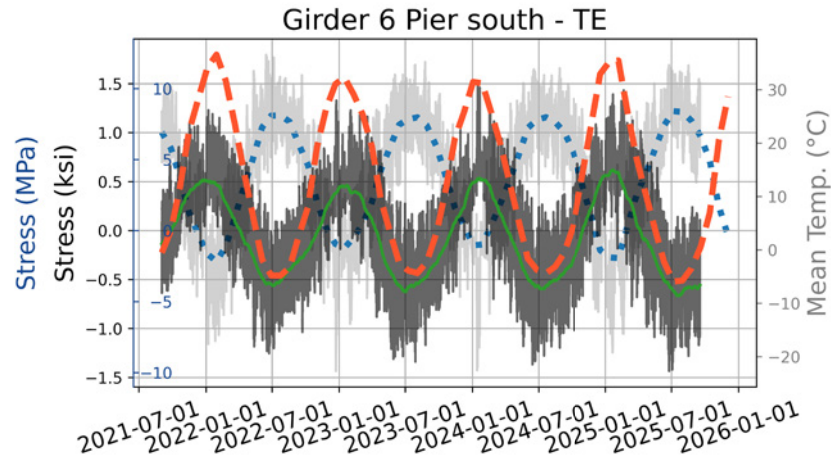
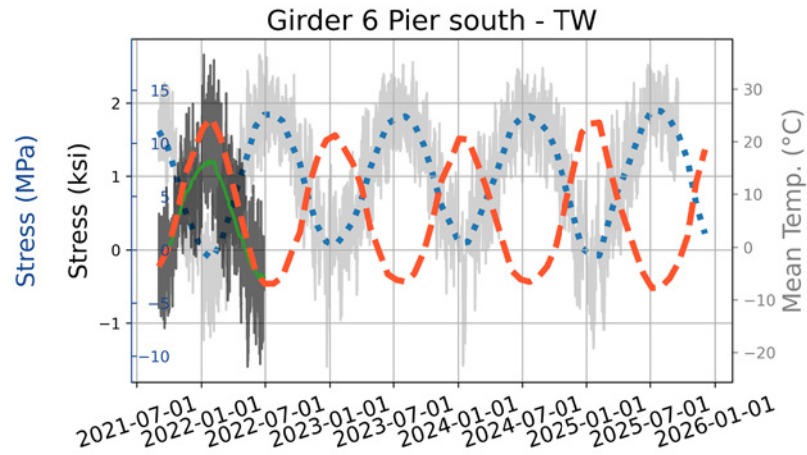


Figure 212. Graph. Response history stress comparison (FE vs. field data) at Girder 5 Pier North.

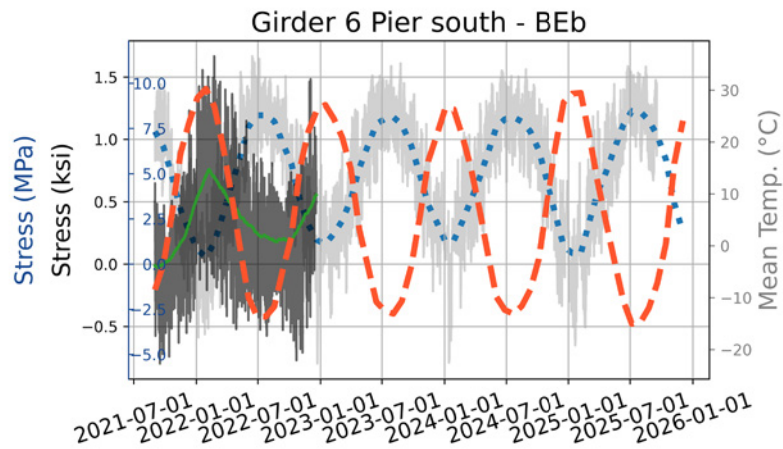
— Field Temp. — Field TE — Field Moving Mean - - FEA TE . . . FEA Temp.



— Field Temp. — Field TW — Field Moving Mean - - FEA TW . . . FEA Temp.



— Field Temp. — Field BEb — Field Moving Mean - - FEA BEb . . . FEA Temp.



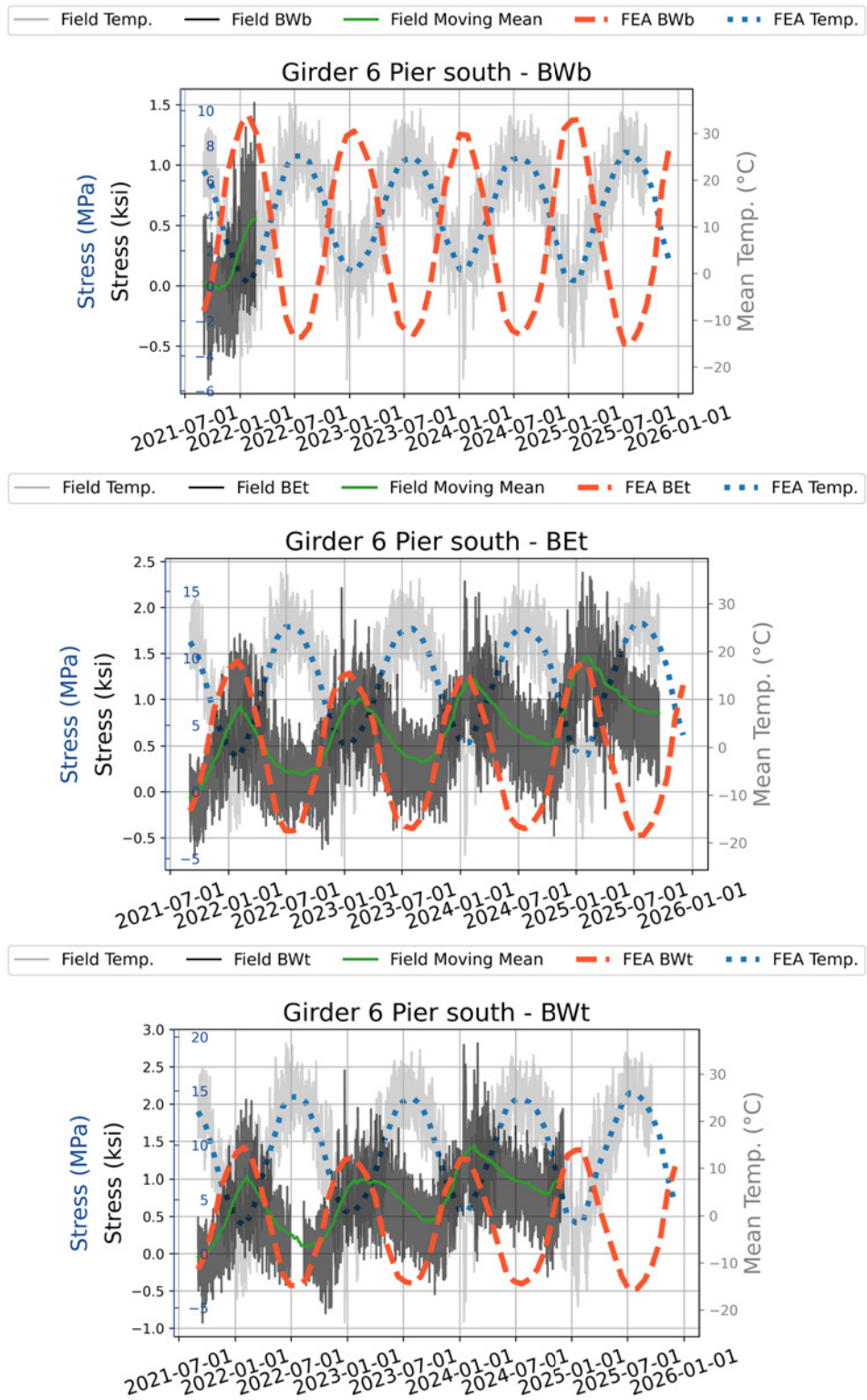


Figure 213. Graph. Response history stress comparison (FE vs. field data) at Girder 6 Pier South.

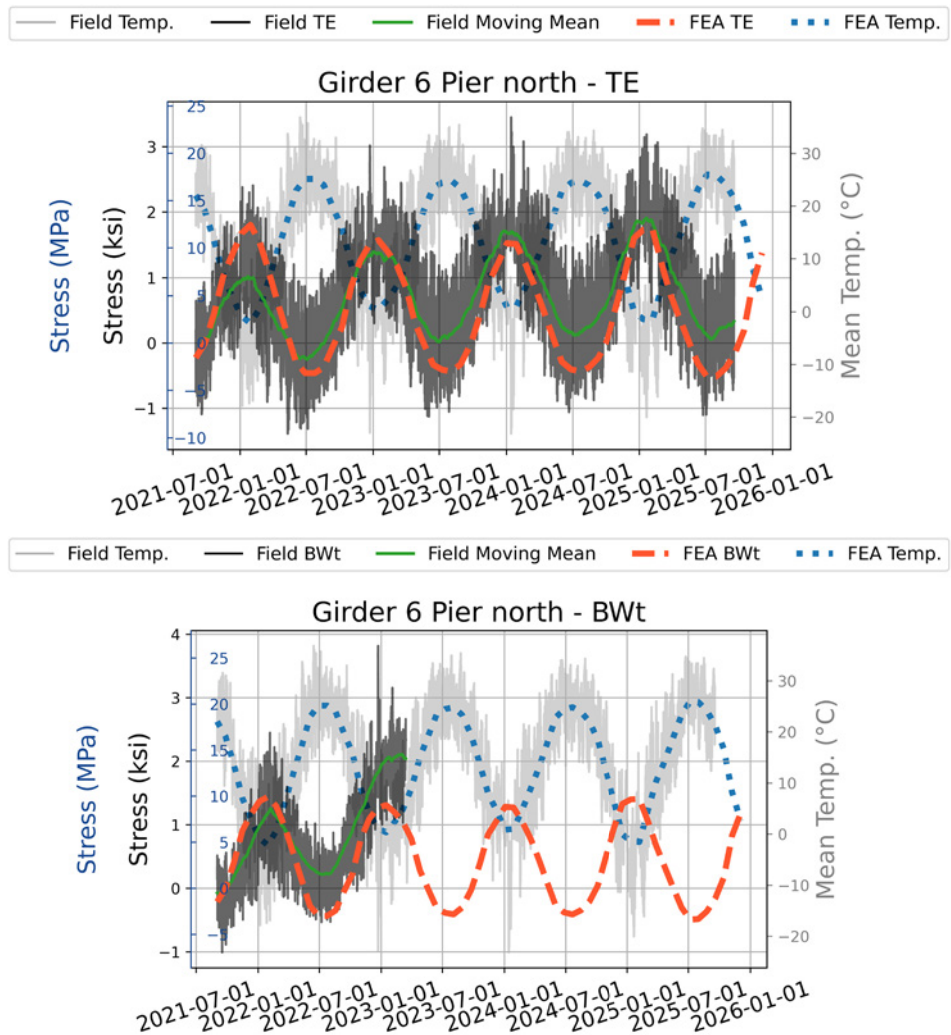


Figure 214. Graph. Response history stress comparison (FE vs. field data) at Girder 6 Pier North.

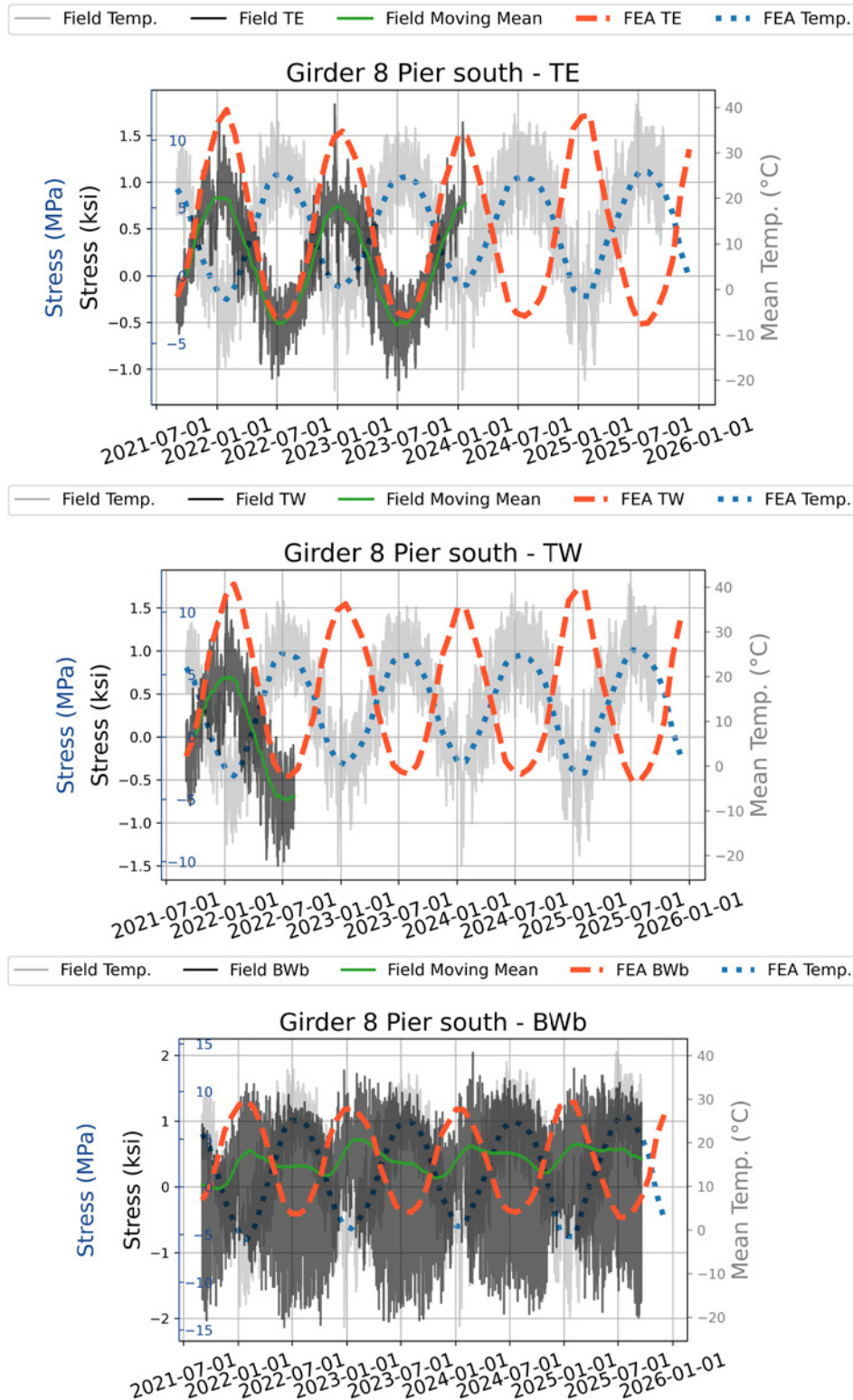
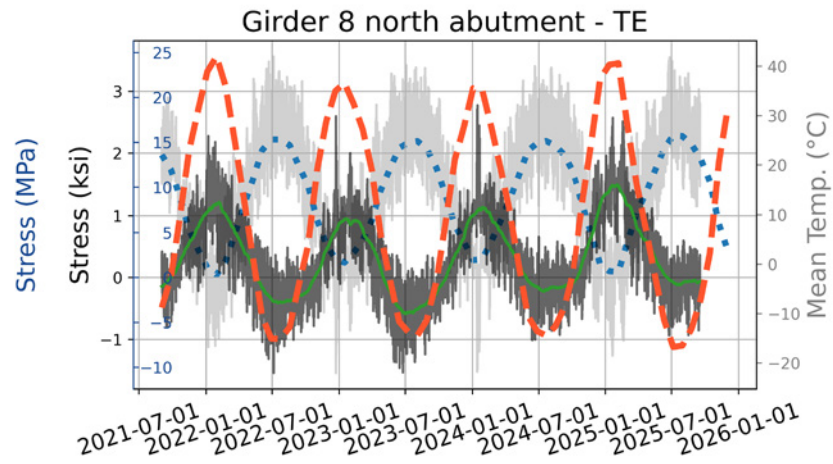
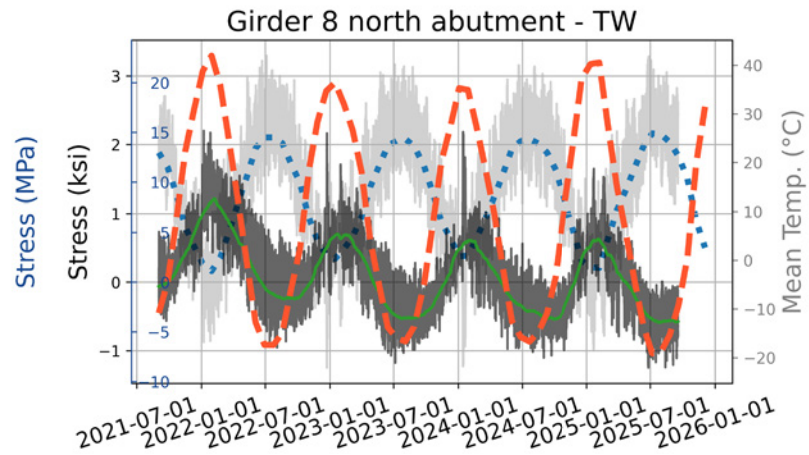


Figure 215. Graph. Response history stress comparison (FE vs. field data) at Girder 8 Pier.

— Field Temp. — Field TE — Field Moving Mean - - FEA TE . . . FEA Temp.



— Field Temp. — Field TW — Field Moving Mean - - FEA TW . . . FEA Temp.



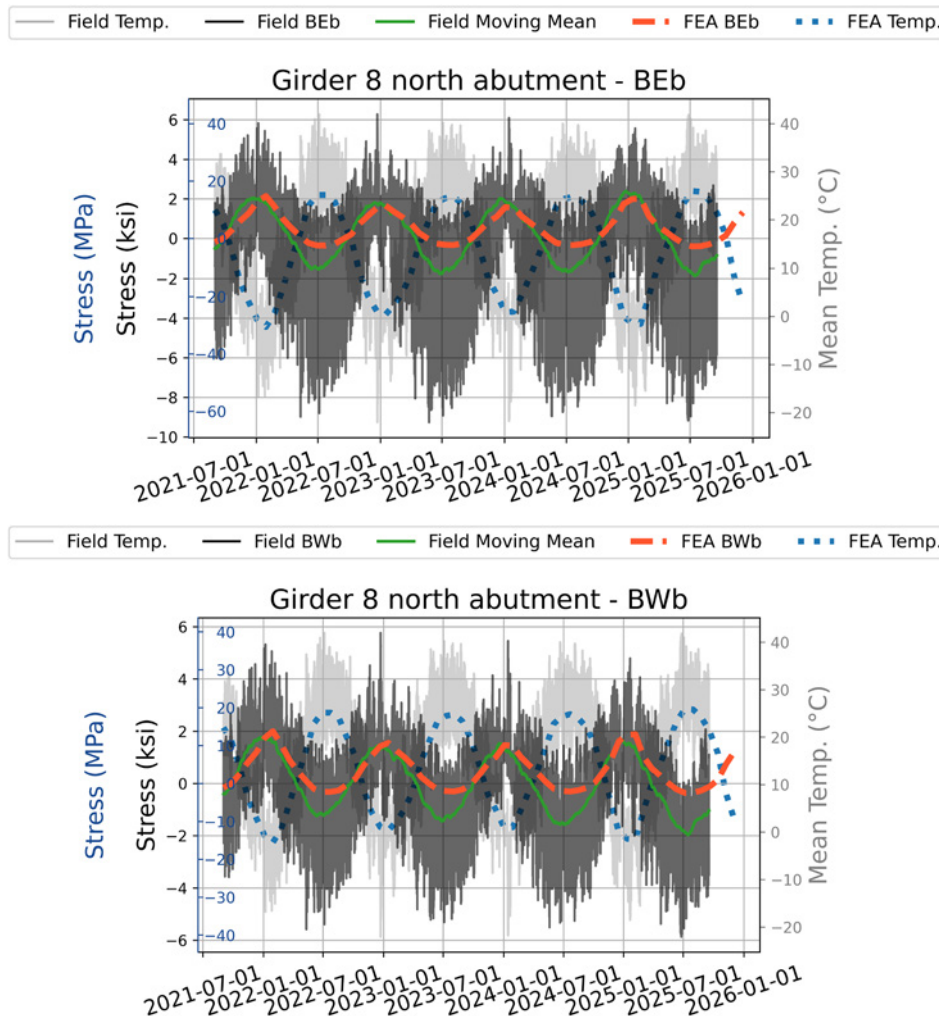


Figure 216. Graph. Response history stress comparison (FE vs. field data) at Girder 8 North Abutment.

DECOMPOSED STRESS DATA FROM GIRDER CROSS-SECTIONS—VALIDATION

Figures 217 through 224 present the comparison between FE-predicted and field-measured decomposed stress components as a function of temperature for all instrumented gauge locations. In these plots, stresses are separated into their contributing components (e.g., strong-axis bending stress, lateral bending stress, axial stress), providing additional insight into the mechanisms governing the structural response. This decomposition enables a more detailed validation of the model by assessing not only the total stress response but also the accuracy in capturing the underlying sources of stress variation.

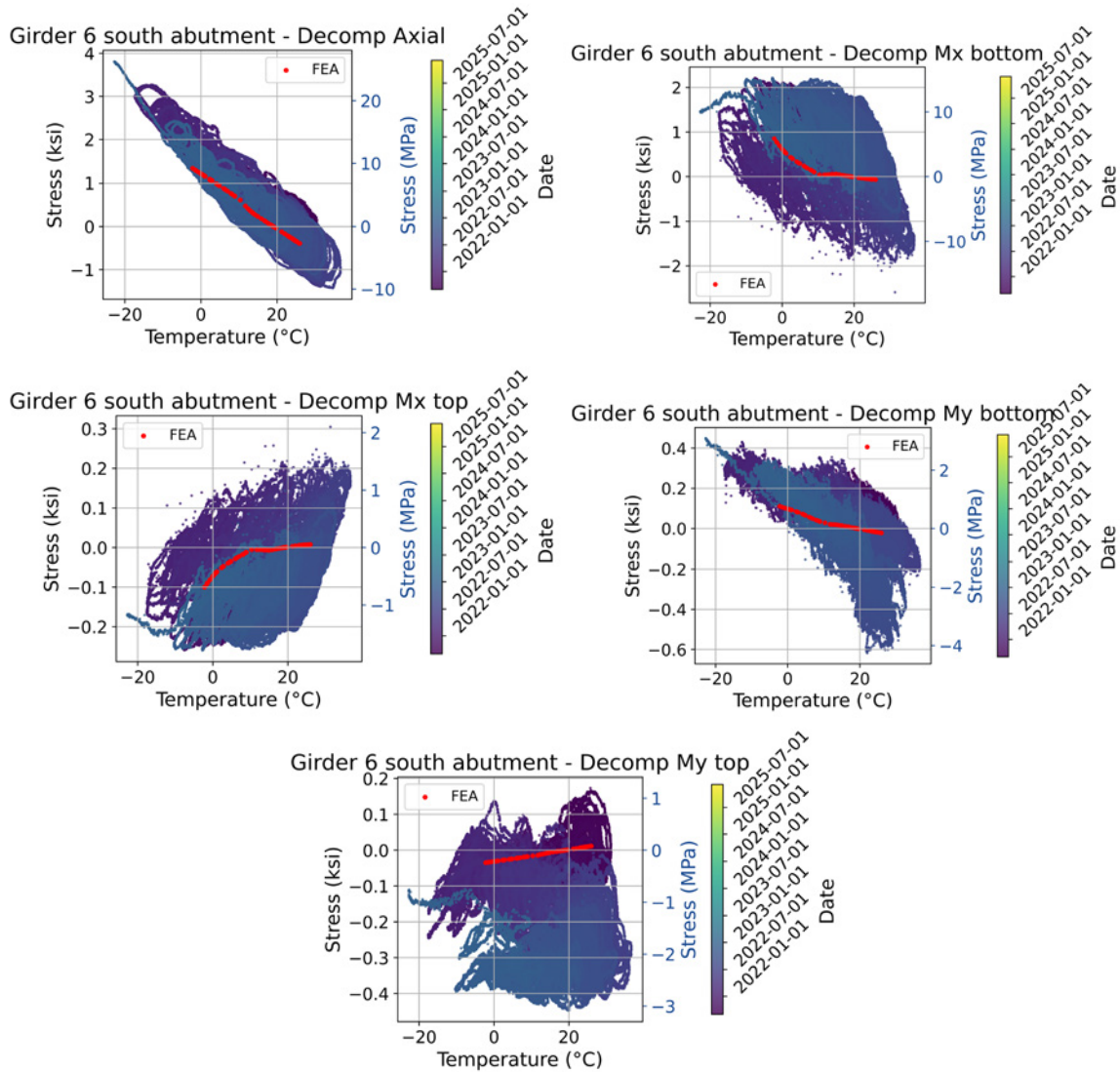


Figure 217. Graph. Temperature-decomposed stress comparison (FE vs. field data) at Girder 6 South Abutment.

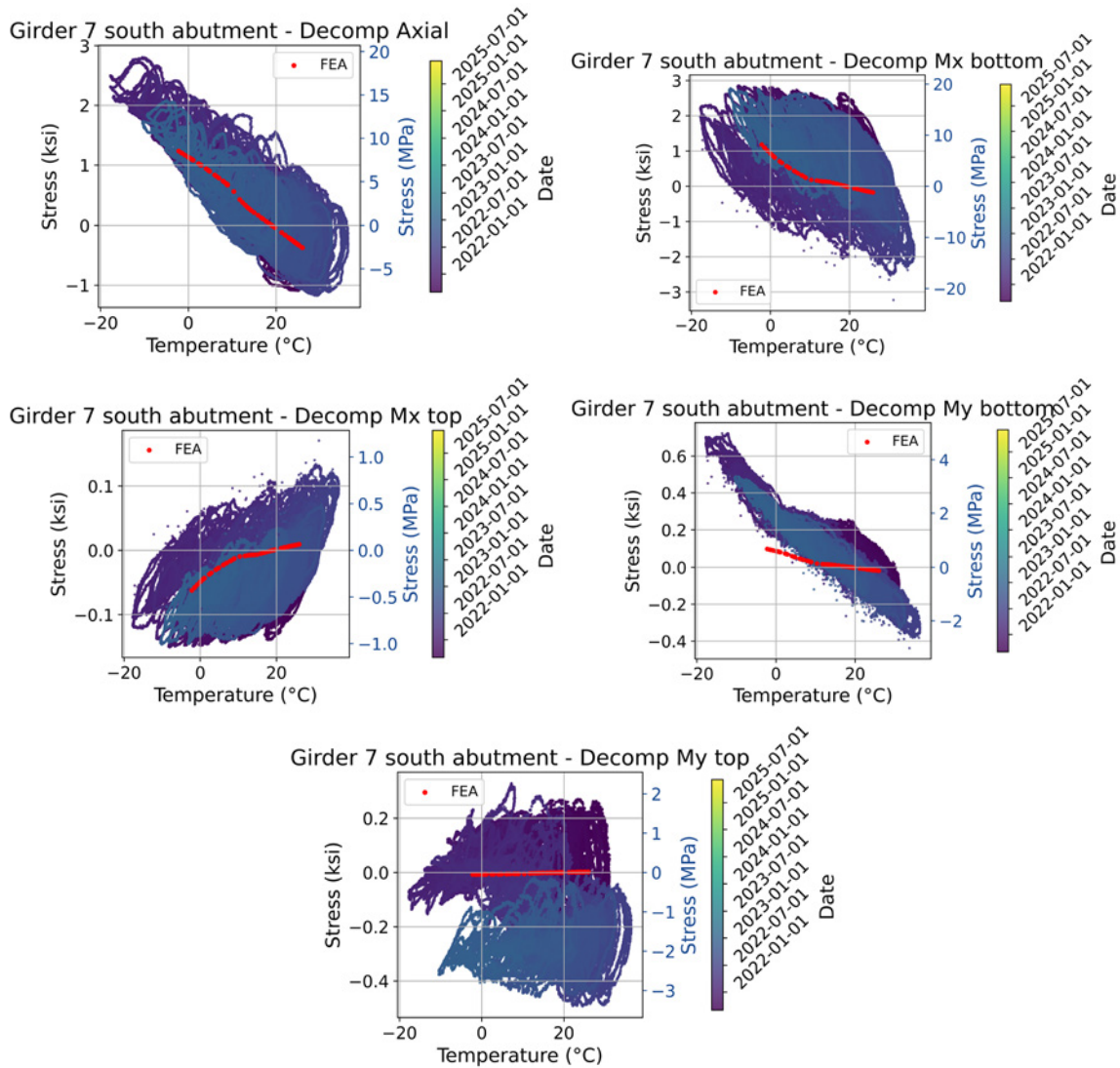


Figure 218. Graph. Temperature-decomposed stress comparison (FE vs. field data) at Girder 7 South Abutment.

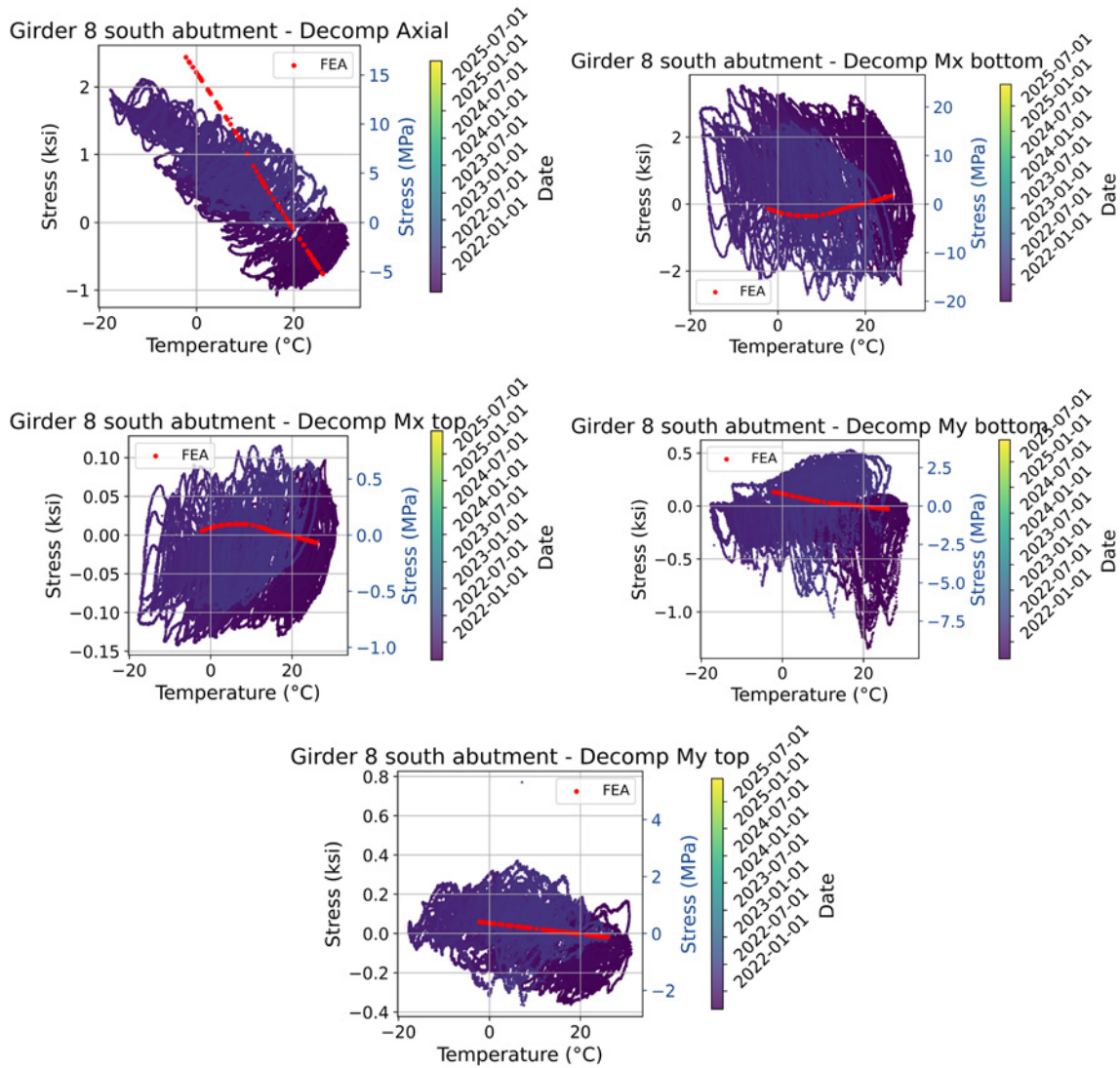


Figure 219. Graph. Temperature-decomposed stress comparison (FE vs. field data) at Girder 8 South Abutment.

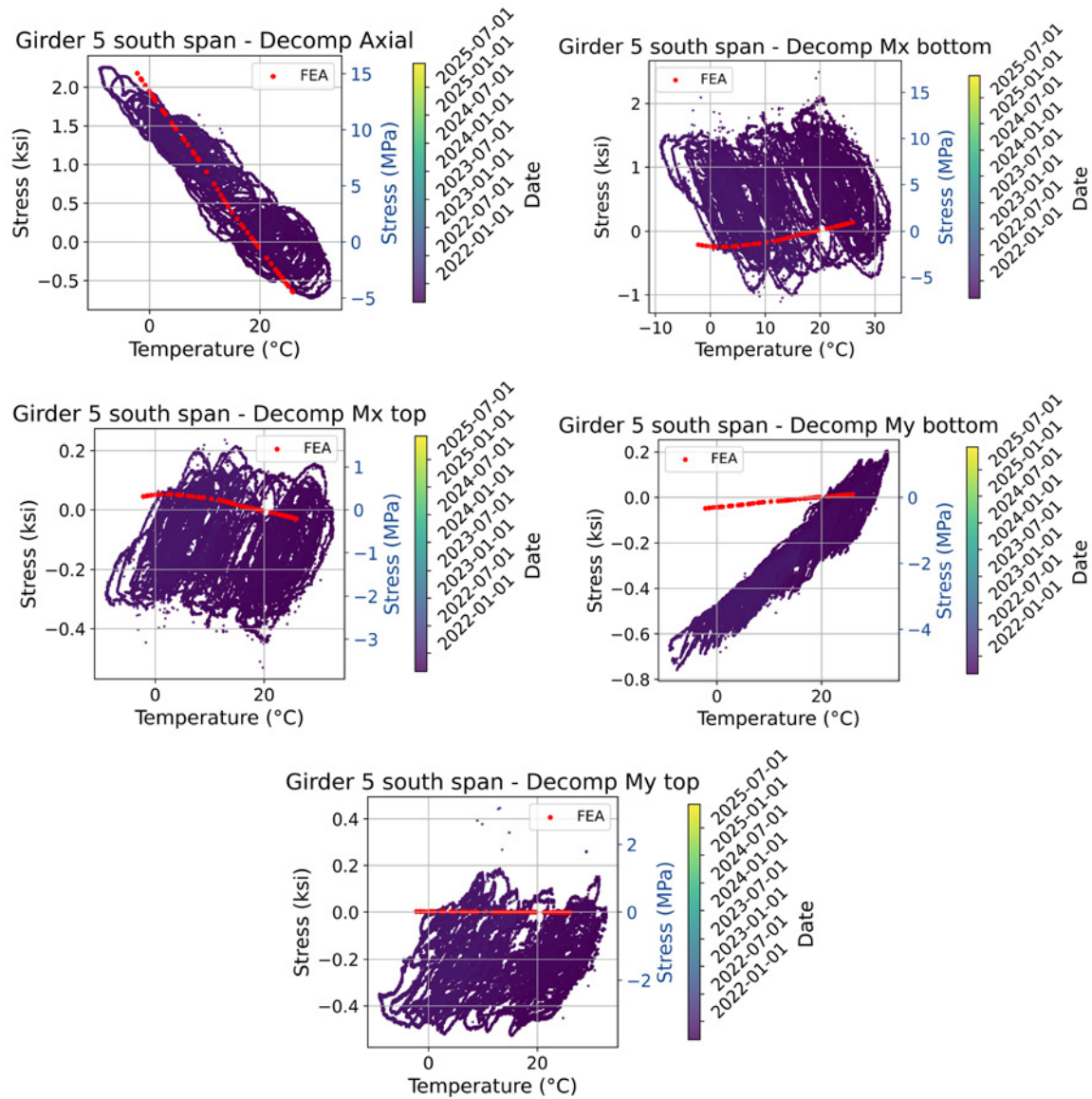


Figure 220. Graph. Temperature-decomposed stress comparison (FE vs. field data) at Girder 5 South Span.

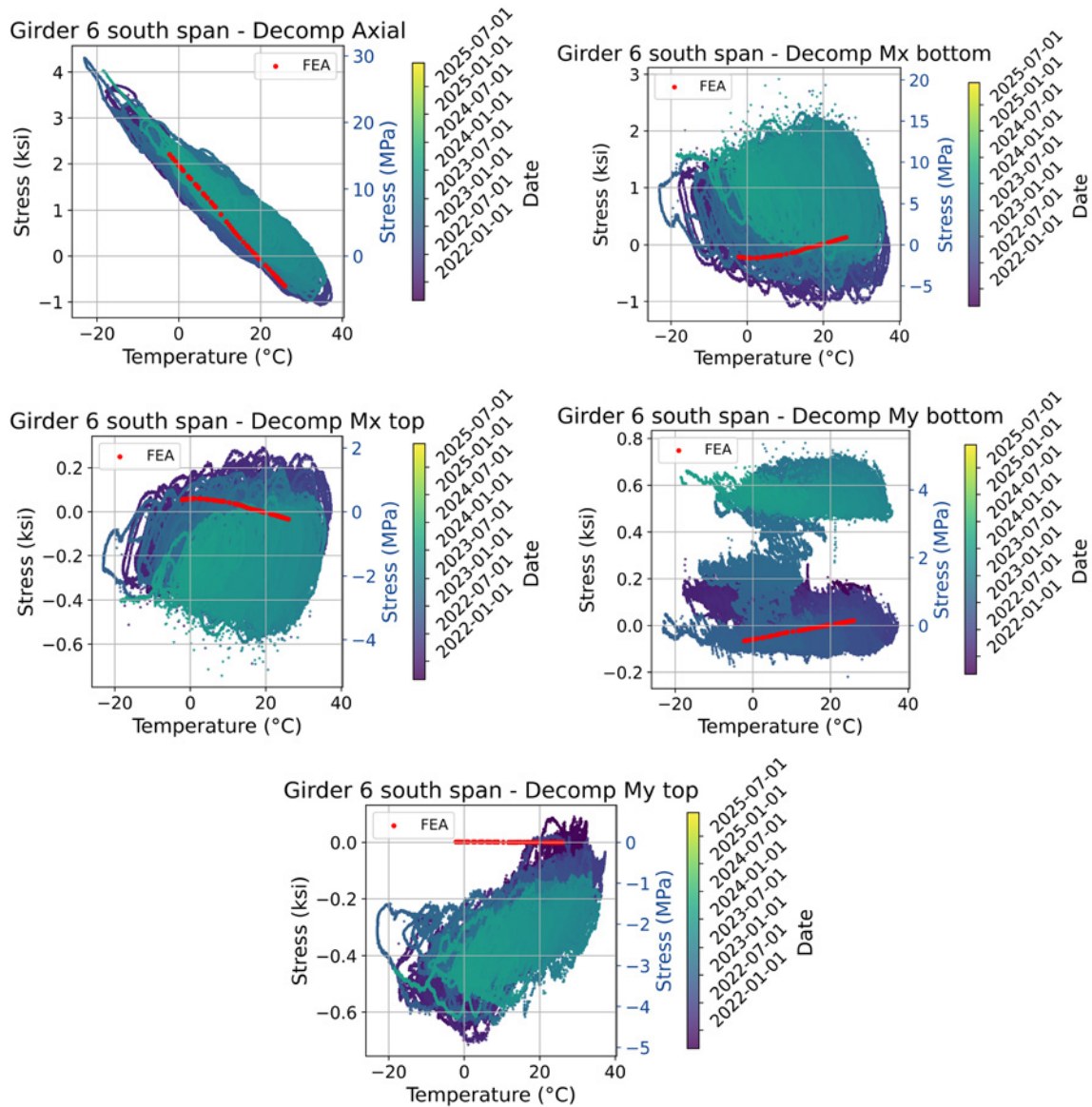


Figure 221. Graph. Temperature-decomposed stress comparison (FE vs. field data) at Girder 6 South Span.

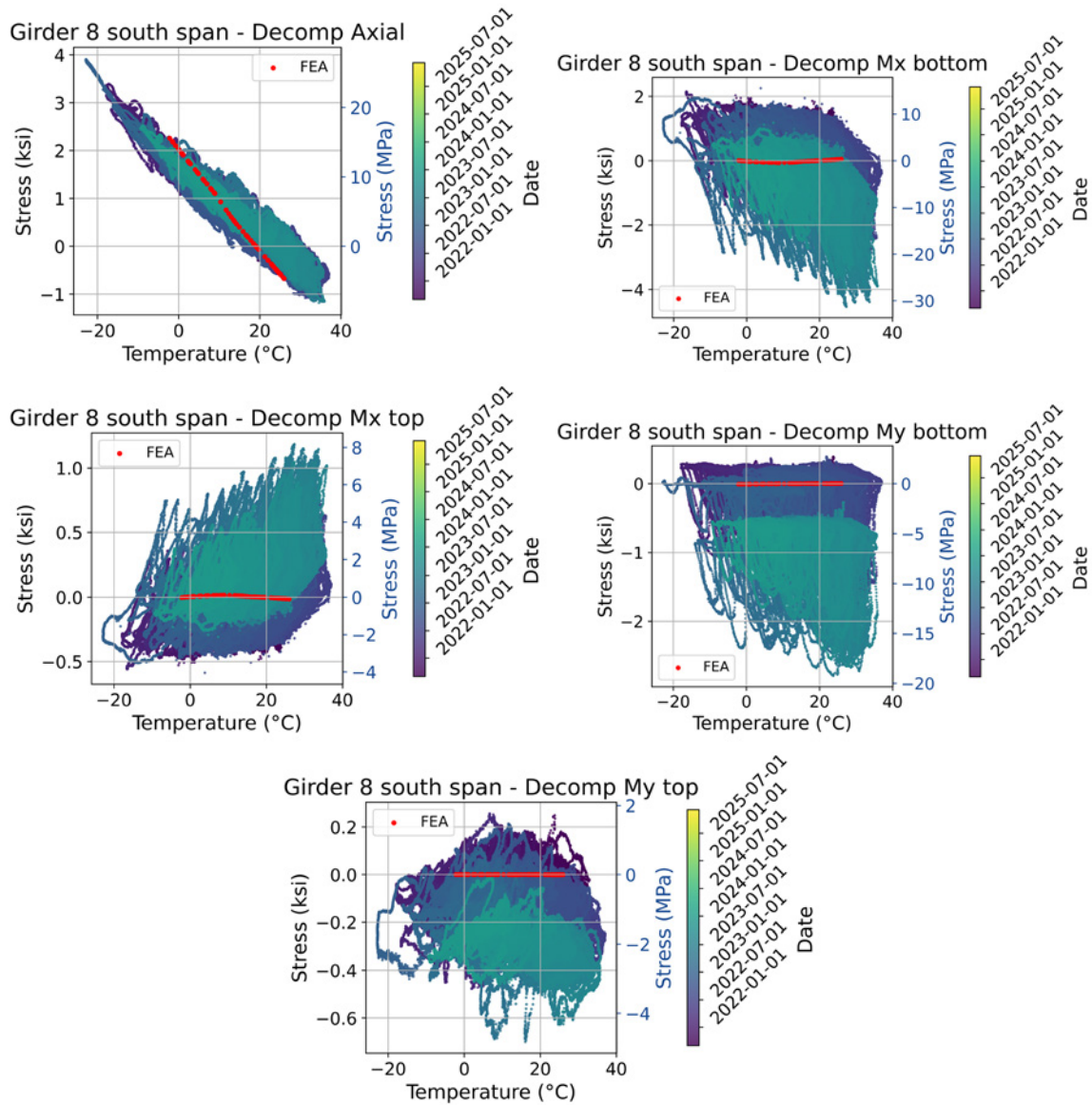


Figure 222. Graph. Temperature-decomposed stress comparison (FE vs. field data) at Girder 8 South Span.

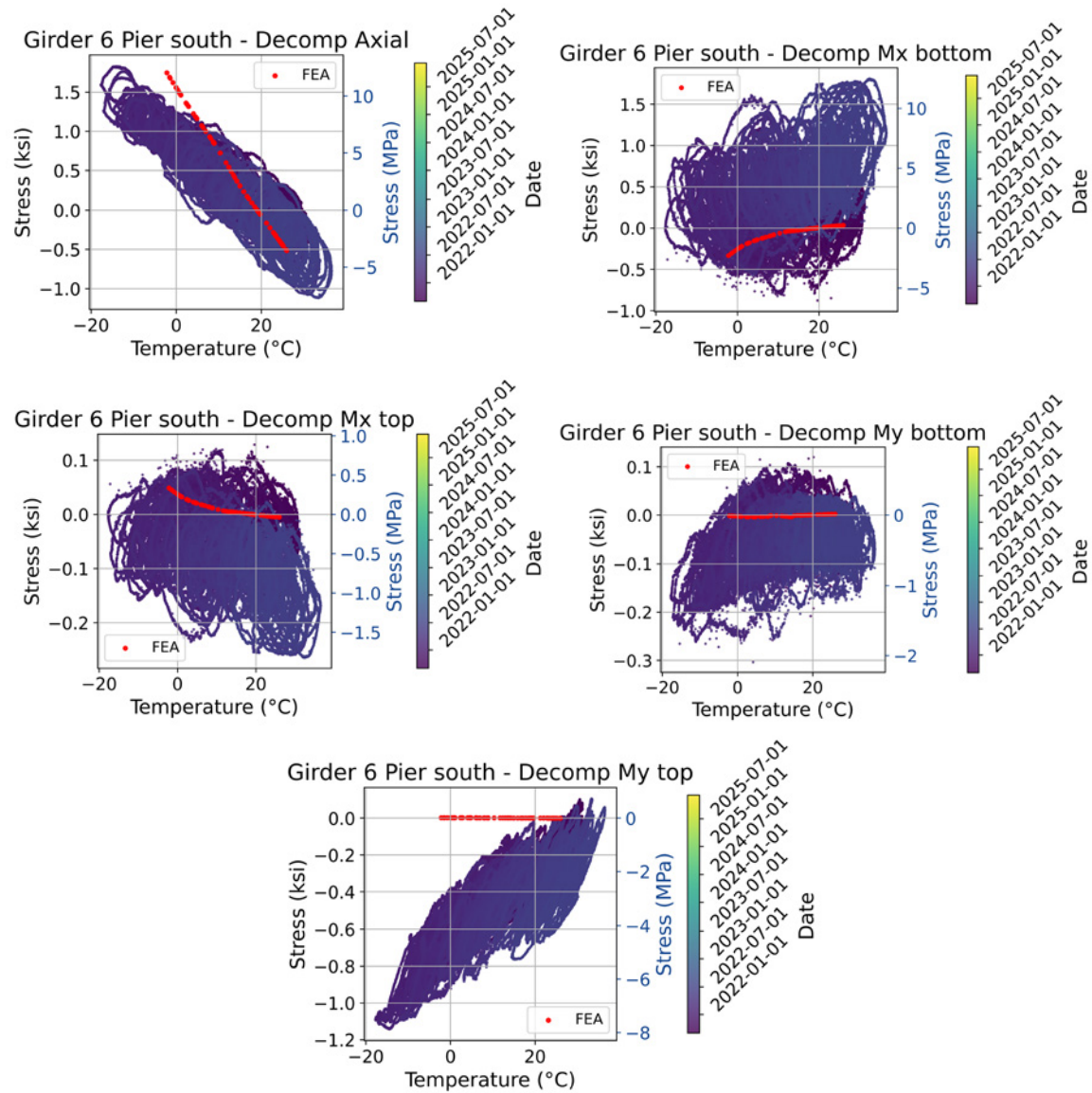


Figure 223. Graph. Temperature-decomposed stress comparison (FE vs. field data) at Girder 6 Pier South.

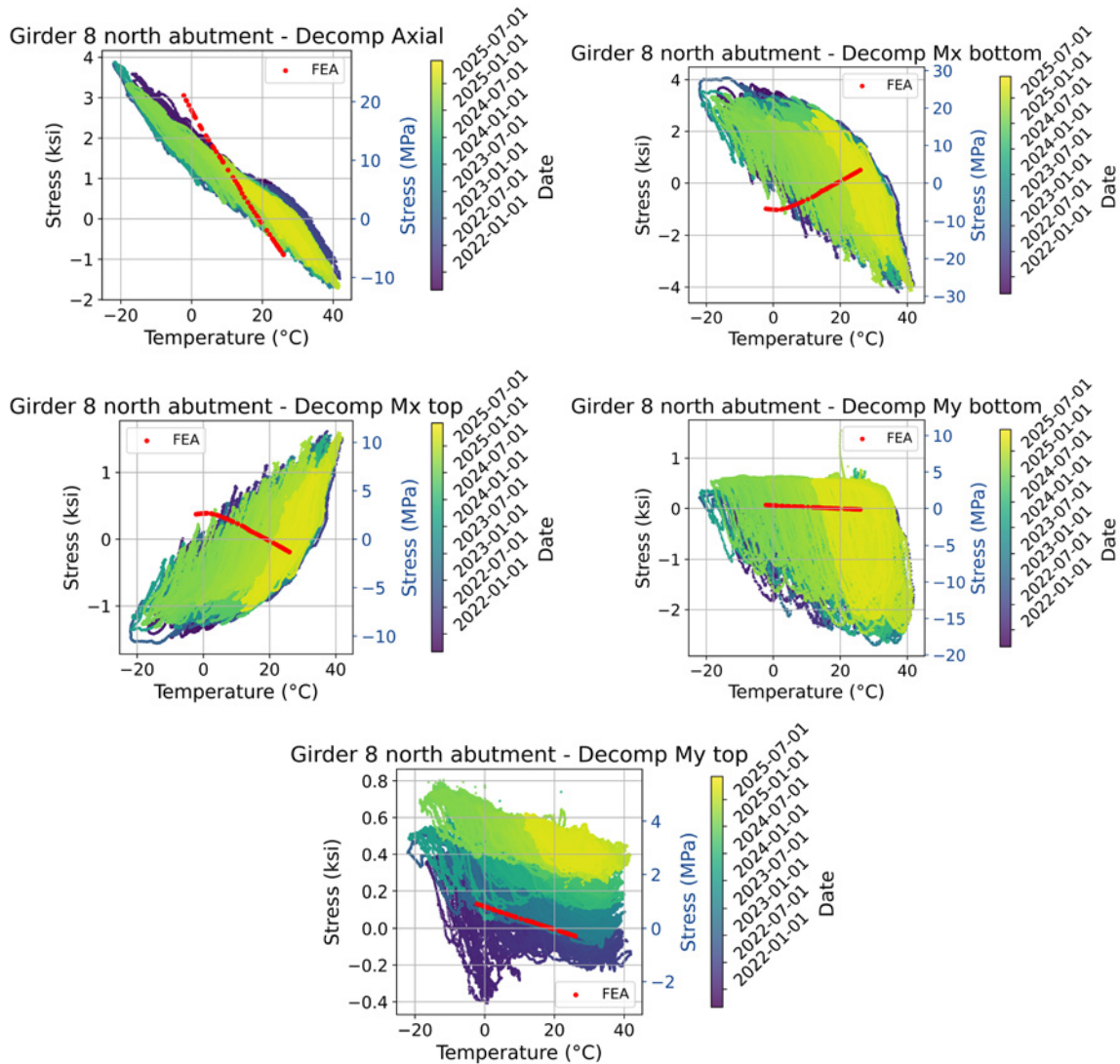


Figure 224. Graph. Temperature-decomposed stress comparison (FE vs. field data) at Girder 8 North Abutment.

CROSS-FRAMES—VALIDATION

Figures 225 through 229 present the comparison between FE-predicted and field-measured responses for the cross-frames as a function of temperature. These plots are used to evaluate the ability of the numerical model to reproduce thermally induced forces and stresses in the cross-frame members, which play a key role in load distribution between girders.

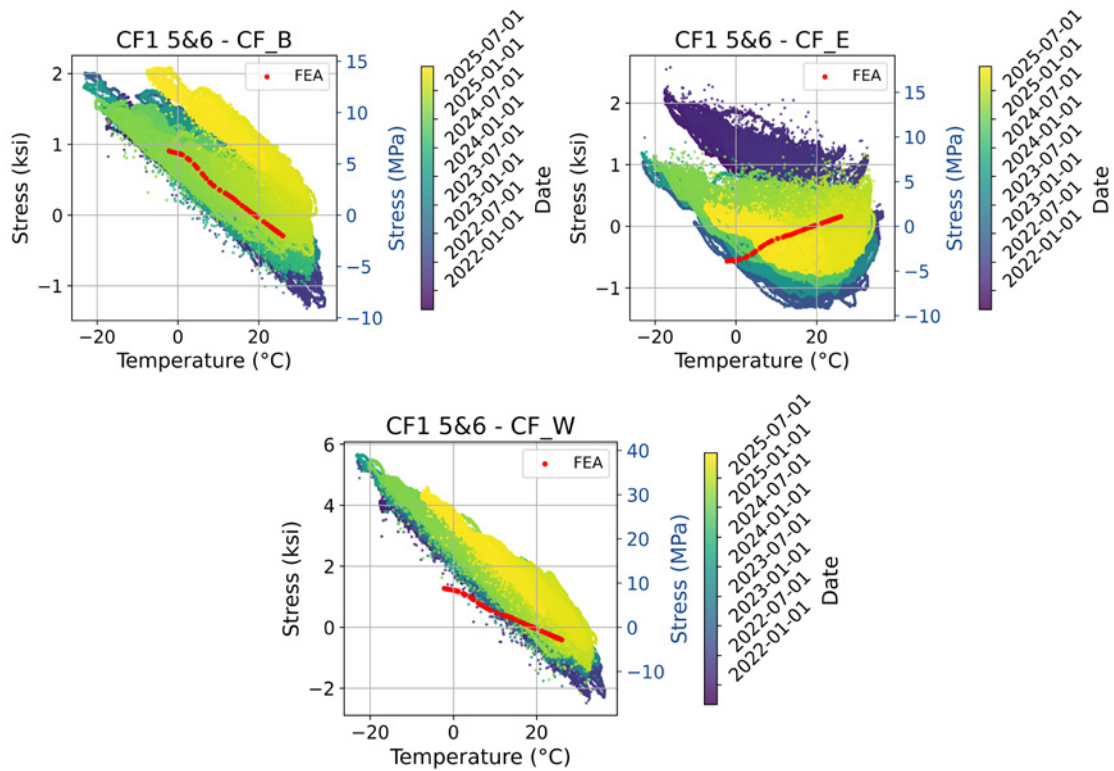


Figure 225. Graph. Temperature-stress comparison (FE vs. field data) at CF1 5&6.

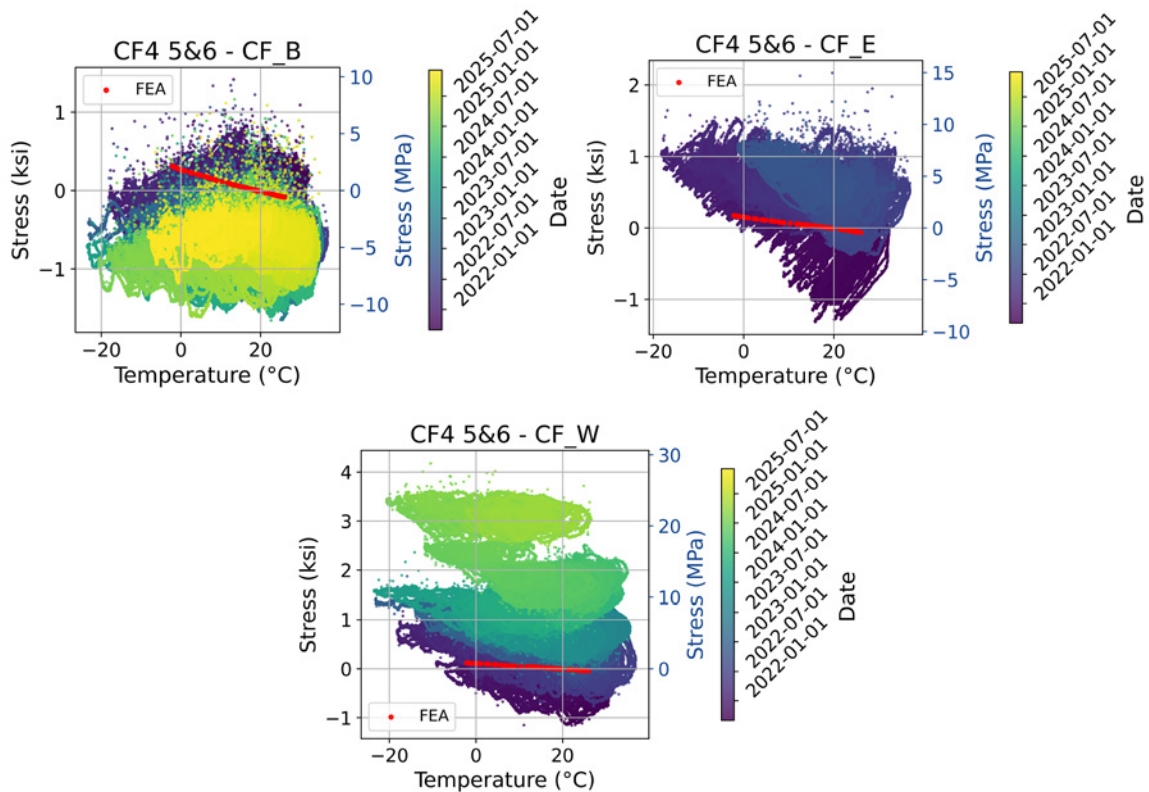


Figure 226. Graph. Temperature-stress comparison (FE vs. field data) at CF4 5&6.

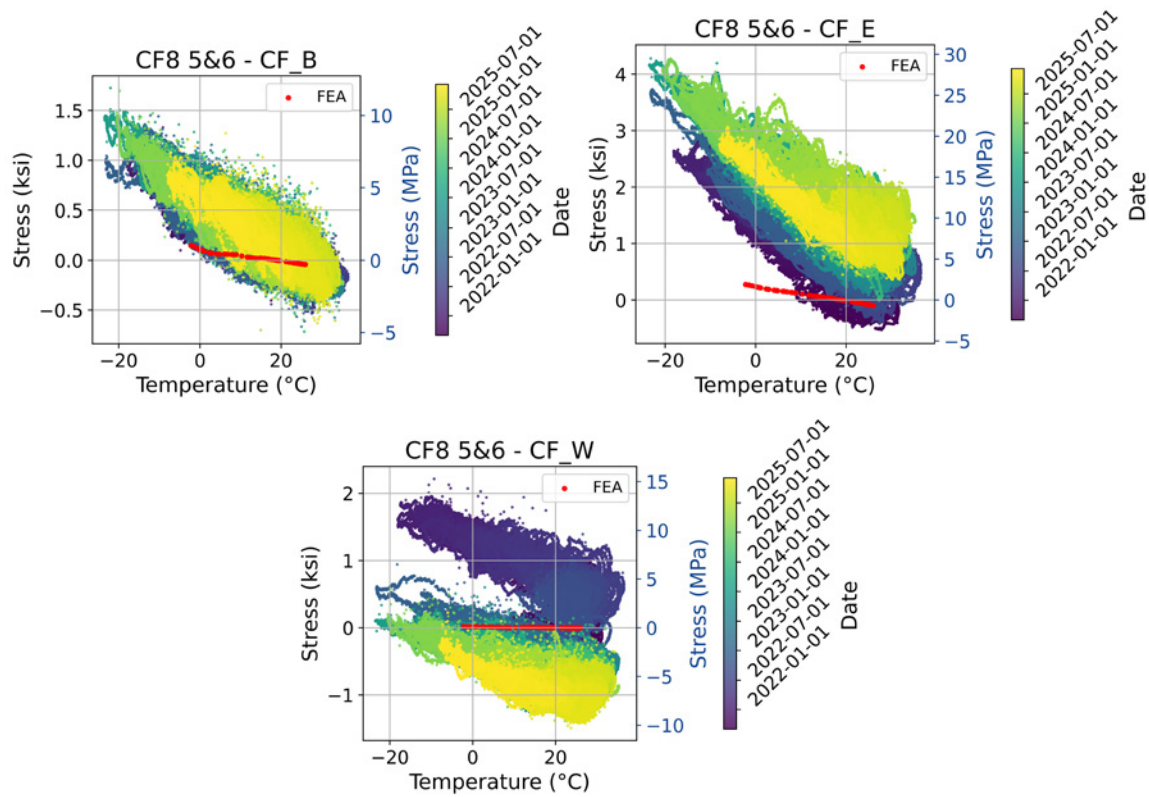


Figure 227. Graph. Temperature-stress comparison (FE vs. field data) at CF8 5&6.

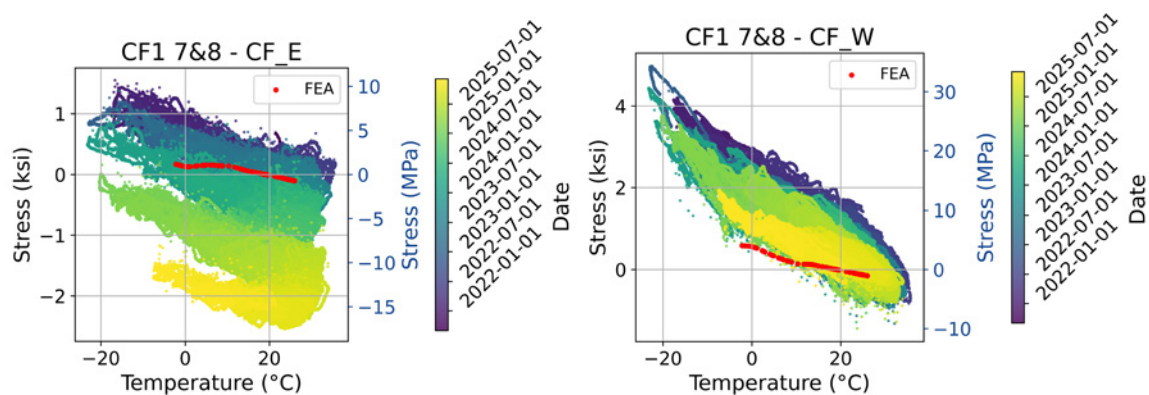


Figure 228. Graph. Temperature-stress comparison (FE vs. field data) at CF1 7&8.

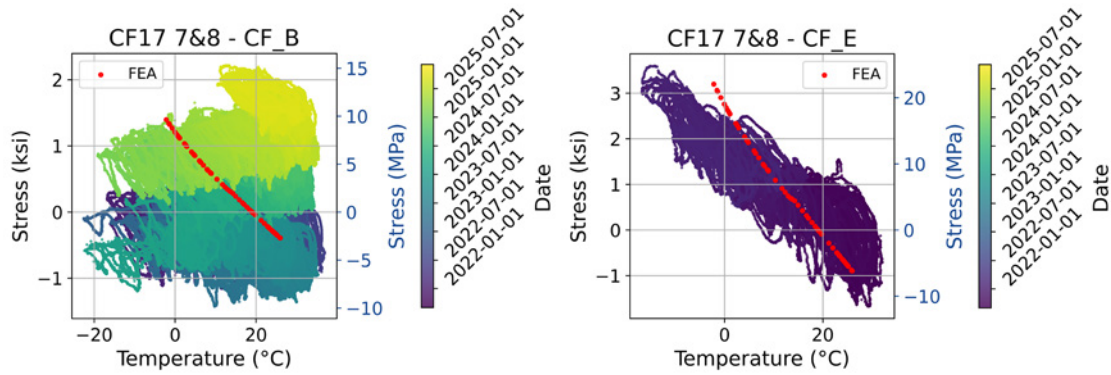


Figure 229. Graph. Temperature-stress comparison (FE vs. field data) at CF17 7&8.

CRACKMETERS—VALIDATION

Figure 230 and Figure 231 present the comparison between FE-predicted and field-measured crackmeter responses as a function of temperature. These measurements capture relative displacements at critical locations and are used to assess the model’s ability to reproduce global bridge movements driven by thermal effects. The comparison provides a direct validation of deformation patterns predicted by the FE model.

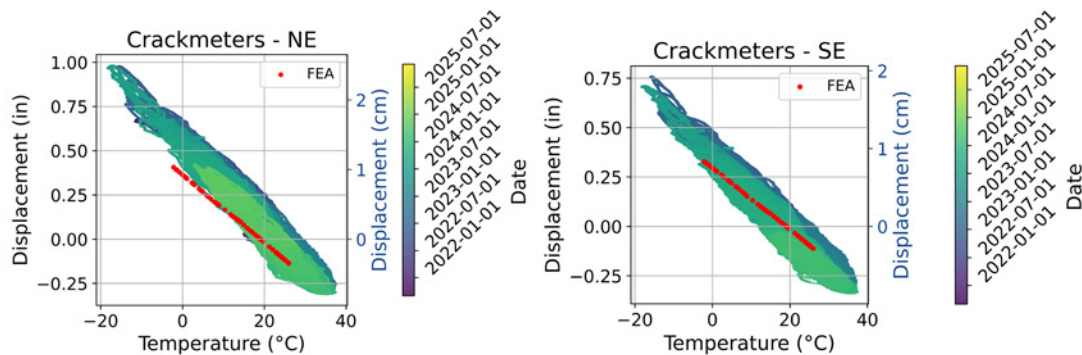


Figure 230. Graph. Temperature-displacement comparison (FE vs. field data) at east crackmeters (parallel).

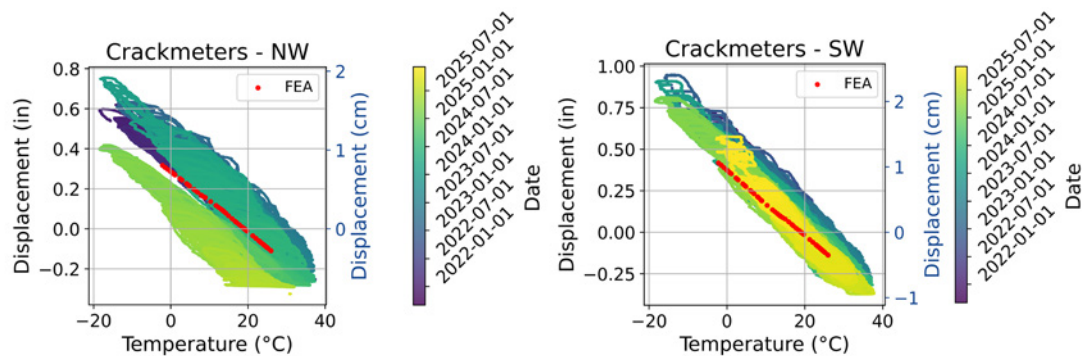


Figure 231. Graph. Temperature-displacement comparison (FE vs. field data) at west crackmeters (parallel).

TILTMETERS—VALIDATION

Figure 232 presents the comparison between FE-predicted and field-measured tiltmeter responses as a function of temperature. These plots are used to evaluate the model’s ability to capture girder rotations associated with thermal loading.

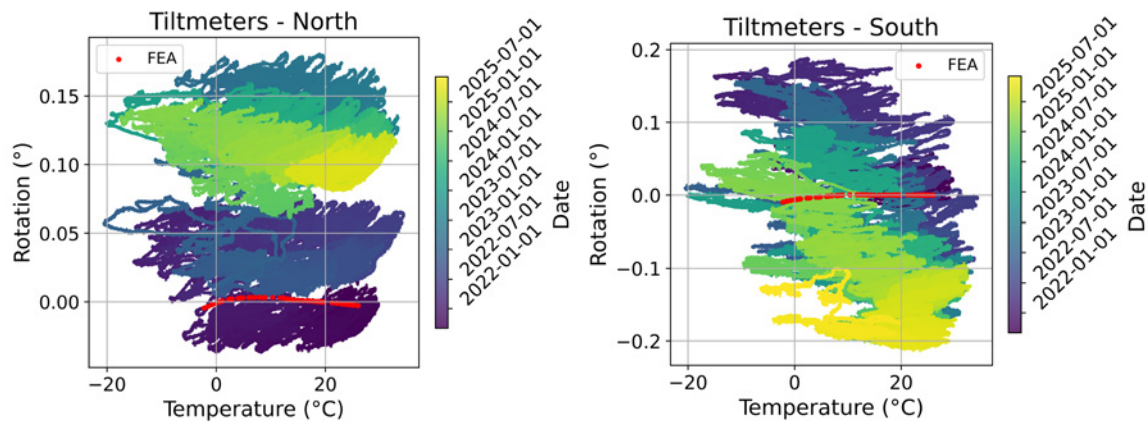


Figure 232. Graph. Temperature-rotation comparison (FE vs. field data) at tiltmeters.



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