BASCULE BRIDGE LIGHTWEIGHT SOLID DECK RETROFIT RESEARCH PROJECT

RESEARCH PROGRAM NOTES

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FLORIDA DEPARTMENT OF TRANSPORTATION

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ALUMINUM ORTHOTROPIC DECK
RESEARCH PROGRAM NOTES

1.0 INTRODUCTION

The purpose of this document is to describe the research and development of a new 5-inch deep aluminum orthotropic deck system developed by the Florida Department of Transportation in consultation with AlumaBridge, SAPA, et al. The deck system was developed specifically to replace 5-inch deep steel open grid deck on typical Florida bascule bridges on an approximately weight neutral basis.

The new deck system is a derivative of an 8-inch deep aluminum orthotropic deck system originally developed by Reynolds Metals Company in the mid 1990's. This earlier deck system was the subject of a laboratory and in-service field testing program sponsored by FHWA and Virginia Department of Transportation and performed by Virginia Tech[1],[2]. The deck system was initially installed on two bridges including Route 58 over Little Buffalo Creek near Mecklenburg, Virginia and Corbin Bridge near Huntingdon, Virginia.

The aluminum orthotropic deck system received renewed interest following the acquisition of Reynolds Metal Company by SAPA and the development of friction stir welding (FSW) technology. FSW addresses many of the concerns with the previous metal inert gas (MIG) welding of extrusions including improved weld quality and reduced cost. The 8-inch deep aluminum orthotropic deck with FSW was recently installed on a bridge in Sandisfield, Massachusetts and St. Ambroise River Bridge, Ontario, Quebec. There are several proposals for additional installations in the United States and Canada.

Although the new 5-inch deep deck system is a derivative of a previously tested deck system, there are a number of differences between the proposed design and the earlier design that warrants additional research and testing before the deck system is placed into service. This document describes the research performed to date and details of first phase of the proposed test program.

[1] Dobmeier, Barton, Gomez, Massarelli, and McKeel (1999), *Analytical and Experimental Evaluation of an Aluminum Bridge Deck Panel, Part I: Service Load Performance and Part II: Failure Analysis*
[2] Misch, Barton, Gomez, Massarelli, and McKeel (1999), *Experimental and Analytical Evaluation of an Aluminum Deck Bridge*

1.1 DECK SYSTEM DESCRIPTION

Deck Panels: The new deck system consists of 5-inch deep panels fabricated from a series of closed shape aluminum extrusions (ASTM B221 Alloy 6063-T6) with integrally connected top and bottom plates and series of inclined web members. The configuration of the extrusions has undergone several iterations during development. Current primary extrusions (AlumaBridge Gen II) are 5" deep x 1'-6" or 1'-1½" wide in both "female" and "male" configurations. The primary extrusions include a vertical web and seats at one or both ends that act as built-in weld backing that permits single-sided FSW. End extrusions are 5" deep x 1'-1½" wide. (See Figures 1, 2 and 3) End extrusions finish the ends of the panels and include a lip at the deck top to better retain the wearing surface at the panel edge, and a lip to retain an expansion joint seal. End extrusions also provide a means for varying the width of the panel up to 4½" by trimming the top and bottom flanges. (See Figure 4) The two primary extrusion widths and variable width end extrusions allow for panels of any width. The individual extrusions are spliced together using single-sided FSW to create deck panels. (See Figure 4) Extrusion trials confirm that the proposed profiles can be extruded to maximum lengths of 32.0'. FSW limits the width of panels to 14.0'. The unit weight of the deck without wearing surface and fasteners is approximately 17.4 psf, although weight can vary slightly depending on panel widths. (See Figure 5 for Deck Section Properties)

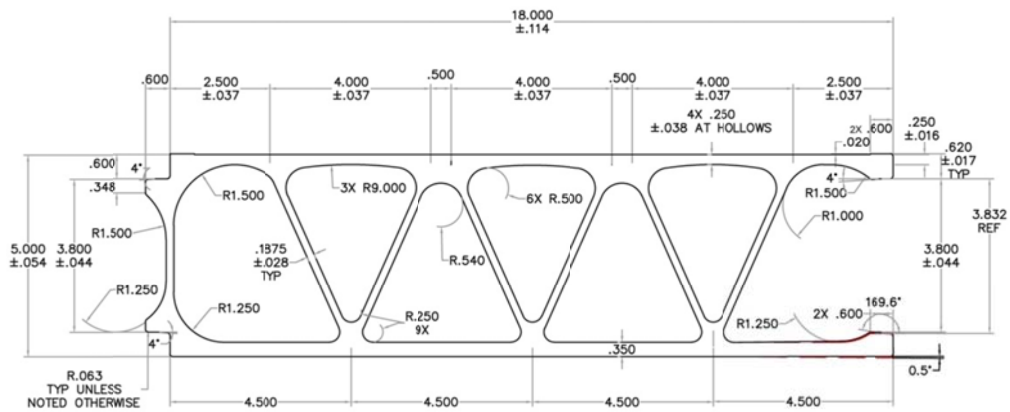


Figure 1 - 5-inch Gen II Female Primary Extrusion

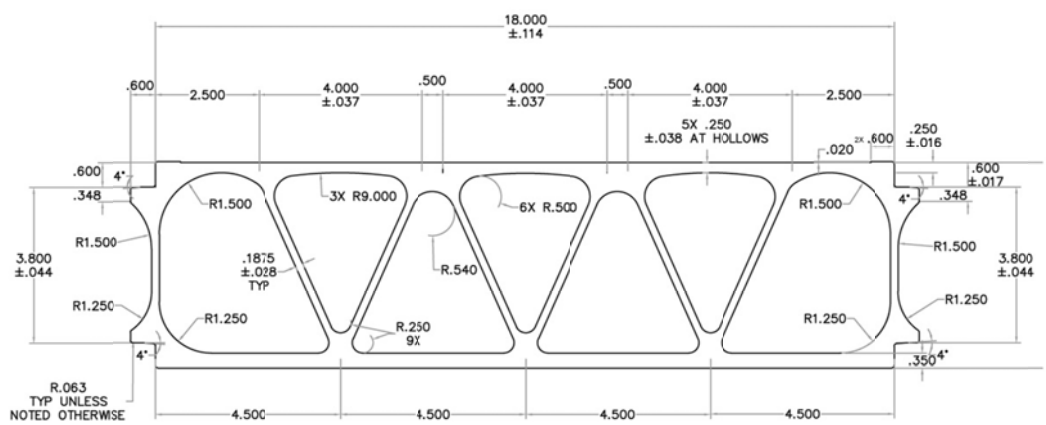


Figure 2 - 5-inch Gen II Male Primary Extrusion

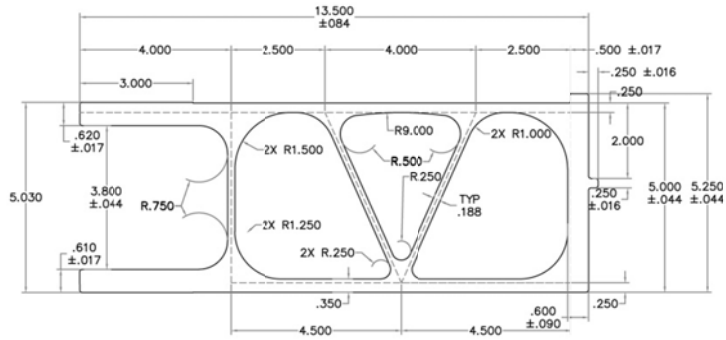


Figure 3 - 5-inch End Extrusion

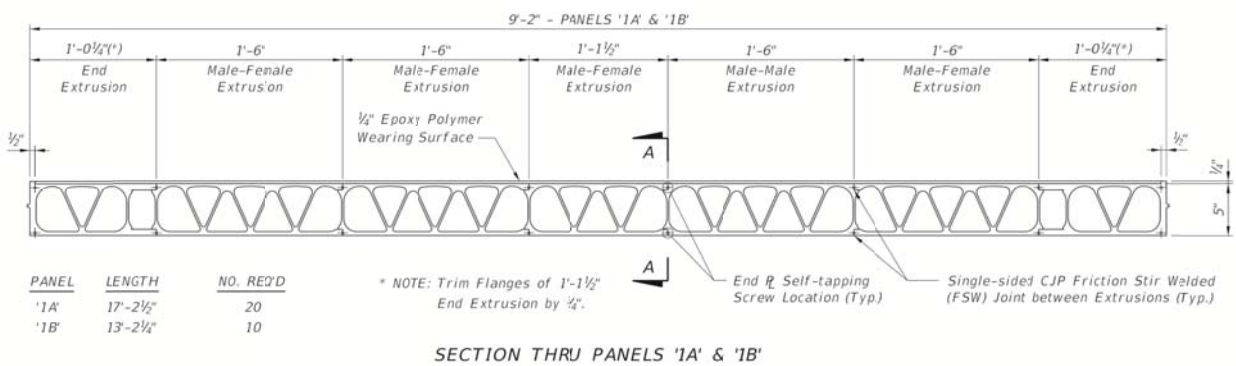


Figure 4 - Typical Deck Panel

TABLE 2 - DECK SECTION PROPERTIES

PARAMETER	VALUE
Cross Section Area, A	14.29 in2/ft
Moment of Inertia, Ix	58.09 in4/ft
Neutral Axis Ref., ybott	2.53 in
Neutral Axis Ref., ytop	2.47 in
Section Modulus, Sxbott	23.49 in3/ft
Section Modulus, Sxtop	22.99 in3/ft
Weight (w/o Wear. Surf.)	17.4 psf
Weight (w/ Wear. Surf.)	20.9 psf

NOTE: Minimum Section Properties conservatively based on 1'-6" Wide Male-Female Extrusions.

Figure 5 – Deck Section Properties

Support Framing: The deck panel design has been developed specifically to replace 5" deep steel open grid deck in typical stringer and floorbeam steel framing systems commonly found on bascule bridges. Similar to steel open grid deck, the aluminum orthotropic deck panels are supported on top of the stringers and span perpendicular to the direction of traffic. Stringers on existing Florida bascule bridges are typically spaced from 3.5' to 5.0' on center. Preliminary analysis of the new deck system indicates that the stringers can be spaced at a maximum spacing of 6.0' with a maximum deck transverse cantilever of 2.0'. (See Figure 6)

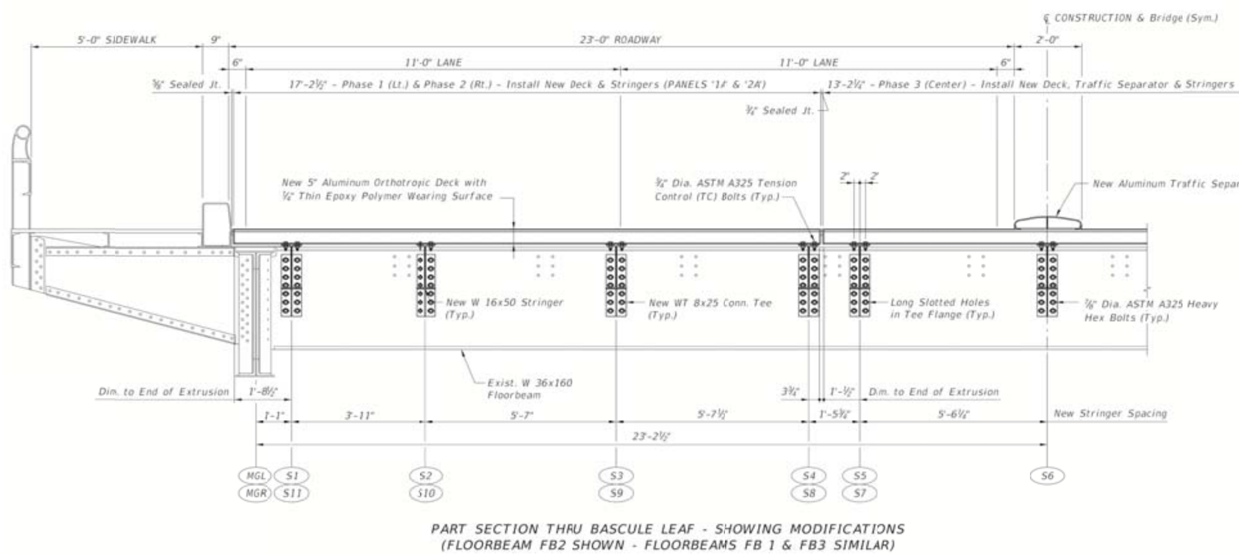


Figure 6 – Typical Bascule Bridge Partial Transverse Section

Replacement of existing stringers is recommended in conjunction with the new deck system to facilitate accelerated bridge construction and reduce the amount of field work required in consideration of the significant number of fasteners used in connecting the deck to the stringers. Replacement of the stringers permits respacing of the stringers to a configuration that avoids support of the deck on the main girders where connection can be difficult.

In order to facilitate pre-bolting of stringers, stringer to floorbeam connections must be detailed such that the stringers clear the floorbeam top flange. (See Figures 7 and 8) Proposed connections utilize a new tee member bolted to the web of the floorbeam with the stem of the tee aligned with the web of the stringer and a pair of connection plates each side of the stringer web/tee stem. The strength and stiffness of the deck panels, which is similar in transverse and longitudinal directions, permits the deck to cantilever in the longitudinal direction from the end of the stringer flange over the floorbeam. This avoids connection to the floorbeam flange where connection can be difficult. The deck and stringer should typically be detailed such that the juncture of the deck inclined webs and bottom plate align with the end of the stringer flange to avoid localized bending of the bottom flange.

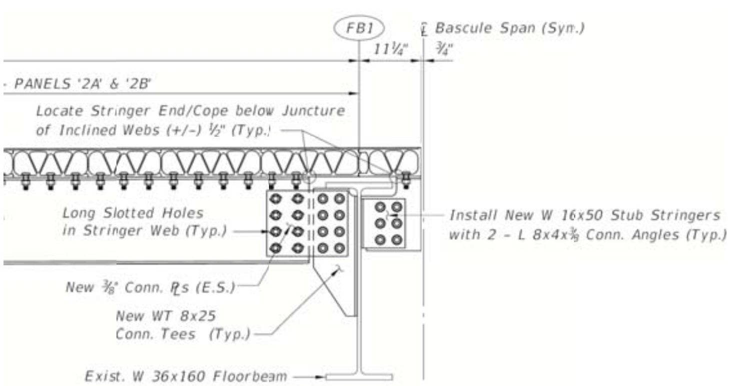


Figure 7 – Typical Bascule Bridge Partial Longitudinal Section at End Floorbeam

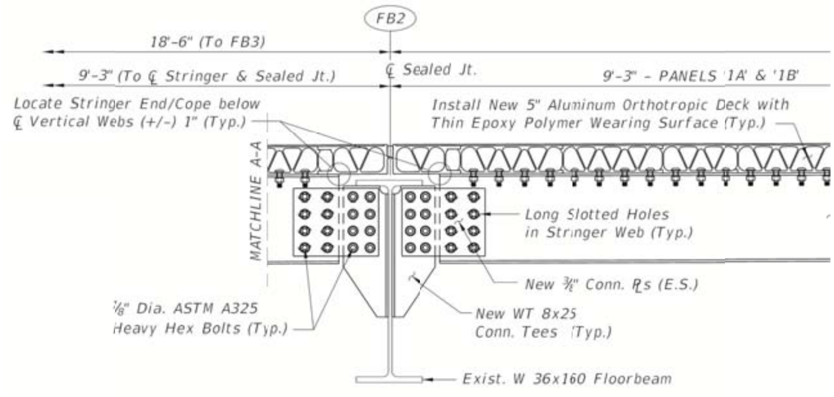


Figure 8 – Typical Bascule Bridge Partial Longitudinal Section at Intermediate Floorbeam

<u>Expansion Joints</u>: The deck system includes transverse expansion joints between individual panels with the joints typically located over the floorbeams and at intermediate points within the stringer span (e.g. near stringer midspan). Longitudinal expansion joints may also be provided between panels where the size of the panels is limited for phased construction and/or shipping considerations. The proposed expansion joints consist of a 1" maximum openings between panels filled with continuous joint filler. The preferred joint filler by the Department is a Watson Bowman, Wabo Evazote Seal (1¼" wide x 2" deep), although other joint fillers such as a backer rod with low modulus joint sealant may also be used. (See Figure 9) The narrowest recommended joint opening between panels to accommodate panel tolerances is ½". Expansion joints are recommended for the following reasons:

- Simplifies panel details by avoiding need for bolted panel splices.
- Expansion joints over the floorbeams accommodate live load deflections and corresponding end rotations of the simple span stringers without restraint from deck continuity.
- Expansion joints spaced at 8.0' to 10.0' intervals mitigate thermal effects in consideration of the difference in coefficients of thermal expansion between aluminum and steel (12.8 x 10^{-6}/F vs. 6.5x10^{-6}/F). The reduced tributary thermal length reduces thermal restraint forces in the stringers and stringer end connections.
- Expansion joints between panels accommodate deck panel fabrication tolerances.

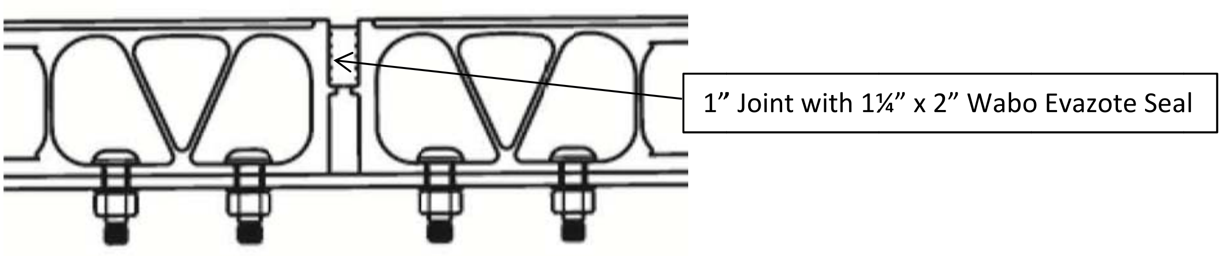


Figure 9 - Expansion Joint Seal

<u>Deck to Stringer Connection</u>: A slip-resistant connection is recommended to resist live loading. Repeated cycles of slip between the deck and stringers are anticipated to cause fretting that might wear protective coatings on the stringers. Slip can be permitted under thermal loading where the number of

cycles of slip is low. To resist slip, deck panels are fastened to the top flange of the stringers with a bolted connection using fully pre-tensioned, conventional 3/4” diameter ASTM A325 bolts of heavy hex (HH) or tension control (TC) type. Bolts, nuts and washers are mechanically galvanized per ASTM B695 Class 50 with nuts over-tapped for fastener assembly and with a lubricant containing a visible dye. A hardened washer shall be provided between the HH nut and aluminum surface. A washer is not required under the head of the TC bolt. Threads are to be excluded from the shear plane. Bolts are to be inserted in standard oversize (15/16” diameter) holes in both the deck plate and stringer flange.

Because aluminum deck panels are hollow, special tooling is required to install and tension the fasteners. With HH type, the nut is located on the interior of the deck and a special tool is required to deliver and temporarily secure the nut and washer while the bolt is tensioned from the exterior using turn-of-nut method. With TC type, the bolt is located on the interior of the deck and special tools are required to deliver and install the bolt in the hole. With TC type bolts, the bolt and nut are held from the exterior and so there is no need for additional means to secure the elements during tensioning. (See Figure 10)

Also considered for future maintenance purposes are 3/4" Lindapter High-Clamping Force Hollo-Bolts (LHB-HCF) (Product Code LHBM20#1 Hexagonal Stainless Steel). The LHB-HCF fastener can be field installed to anchor the hollow aluminum deck panels. The fastener is installed in 1 3/8” diameter standard size holes (1/16” in diameter larger than the split-sleeve). Because sustained pre-tensioning proof loads for this type of fastener is significantly lower than similarly sized conventional ASTM A325 bolts, the bolt is not recommended as the primary fastener type in a slip-resistant connection. The bolt is only recommended in maintenance applications to replace a limited number of conventional ASTM A325 bolts, where required. (See Figure 10)

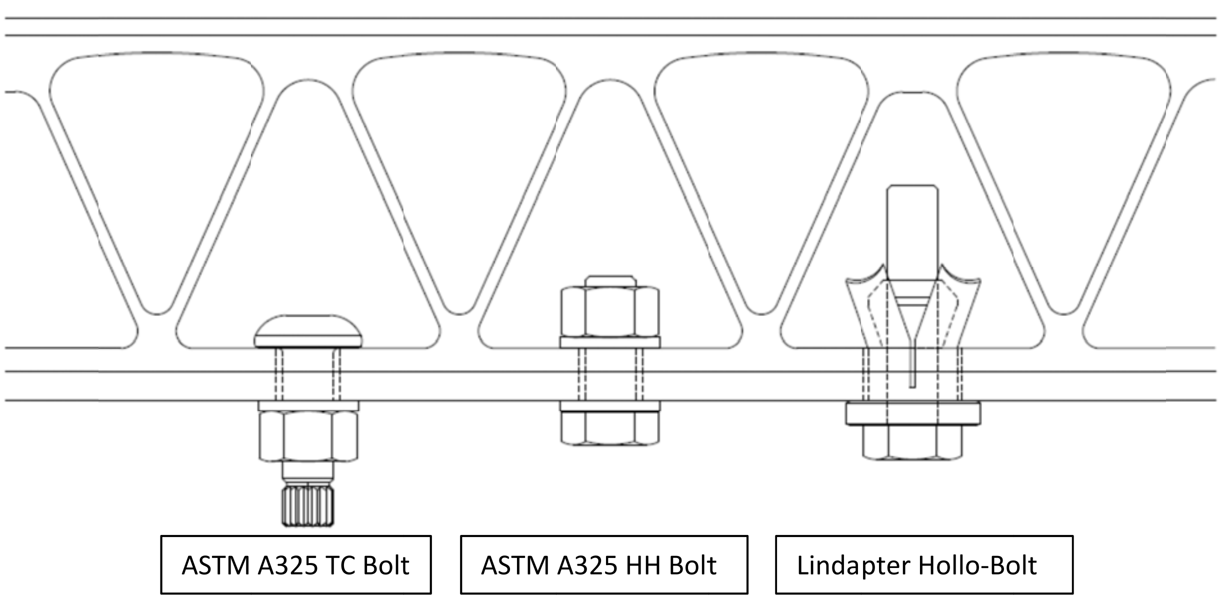


Figure 10 - Fastener Details

The faying surface between the aluminum deck and steel framing have been previously certified by the Research Council on Structural Connections (RCSC) as meeting the requirements for a ASTM A325 Class 'B' (0.5 coefficient of friction). This certification was based on an abrasion blasted aluminum surface per Society for Protective Coatings, SSPC-SP5 (White-Metal Blast Cleaning) to an average substrate profile of 2.0 mils and steel members containing either hot-dip galvanized coating in accordance with ASTM D123 or an approved solvent based inorganic zinc primer (e.g. Carboline Carbozinc 11) with 6.0 mils dry film thickness.

<u>Wearing Surface</u>: The aluminum deck receives a skid-resistant wearing surface applied to the top of the panels consisting of two-coats Flexolith (low modulus epoxy coating system manufactured by Euclid Chemical Company) and a broadcast overlay (1/4" thickness with unit weight of 3 to 4 psf.) (See Figure 11) Three-coats (3/8" thickness with unit weight of 5 to 6 psf) can be used to achieve a longer service life on bridges that can support the additional weight.

Figure 11 – Wearing Surface

Materials: The Flexolith two-part epoxy resin is applied at a spread rate of 40 to 45 sq. ft/gal in the first coat and 22 to 25 sq. ft/gal in the second coat. The epoxy resin shall meet the following requirements:

PROPERTY	REQUIREMENTS (at 75 +/- 3 deg F and 50% RH)	TEST METHOD
Gel Time	>30 minutes	ASTM C881 Class B (150 g sample)
Tensile Strength (7 day)	2,000 to 5,000 psi	ASTM D638
Tensile Elongation (7 day)	30 – 60 %	ASTM D638
Viscosity (7 day)	1,500 to 2,000 cp	ASTM D2393 (Model RVT Brookfield Viscometer Spindle No. 3 at 20 rpm)
Compressive Strength (24 hr)	5,000 psi	ASTM D695
Part A	9.1 - 9.7 lbs/gal	
Part B	8.0 – 8.6 lbs/gal	

The broadcast overlay includes a basalt aggregate with spread rate of 1.0 to 1.5 lbs/sq. ft in first coat and 1.5 to 2.0 lbs/sq. ft in second coat. Basalt aggregate shall be clean, free of other materials, and meet the following requirements:

PROPERTY	REQUIREMENT
Moisture Content	0.2%
Min. Mohs' Scale Hardness	6
Density (Loose)	94 pcf
Distribution (Sieve Size)	% Weight Passing
# 4	100
# 6	97 - 100
#12	70 - 90
# 20	3 - 20
>#20	0 - 3

Surface Preparation: Prior to applying the wearing surface to panel surfaces, surfaces shall be prepared in an environmentally controlled facility. Panel surfaces shall be abraded with abrasive, non-metallic pad specified for use on aluminum. The abraded surface shall be pressure washed with 5% solution of Chemetall Aluminum NSS cleaner and water heated to 120 – 140 deg F. The pressure washed deck shall be cleaned with pressurized tap water until all soap and suds are removed and cleaning repeated until no water beads on the surface. Panels shall be air dried without application of compressed air. The dried panels shall receive a 15% solution of Chemetall Permatreat 1500 in de-ionized or distilled water using lint free rollers with 3/8-inch or finer nap. Panel surface shall be air dried in clean environment with temperature of 75 – 85 deg F and 40 – 60% relative humidity for a minimum of 24 hours prior to coating application.

Application: Wearing surface shall only be applied by qualified applicator certified by Euclid Chemical Company. Two-part resin shall be mixed per manufacturer's recommendations. Resin shall be applied to panel in clean environment with temperature of 75 – 85 deg F and 40 – 60% relative humidity. Application shall be performed in increments to 1/8" uniform thickness. Aggregate shall be broadcasted to full saturation until no wet spots are visible. Panels shall remain undisturbed for minimum of 24 hours in same controlled environment as application.

Quality Control: A separate aluminum test piece shall be prepared with same wearing surface as the production panel at made at the same time, and under the same environmental, surface preparation, and application conditions as the production panels. The size of each test piece shall be as required to verify bond strength of the wearing surface to the aluminum deck panel in accordance with ASTM C1583. The bond strength of the production panels will be considered adequate if the bond strength for the test piece exceeds 250 psi. Production panels shall be accompanied with a report with the panel identification, inspection date, tested bond strength, and inspector signature certifying adequacy of the test performance.

Previous Testing: The bond durability of the Flexolith wearing surface was previously tested and evaluated for a number of factors including moisture, temperature and applied loading[3].

Future Maintenance: The two-coat wearing surface is anticipated to have a service life of 10 to 15 years and three-coat wearing surface a service life of 15 to 20 years depending on traffic. The original three-coat wearing surface on the Route 58 Bridge over Little Buffalo Creek near Mecklenburg, Virginia is still in service despite 18 years of heavy truck traffic and snow plowing.

It is recommended that the wearing surface be resurfaced before it is worn to the depth of the aluminum substrate. Resurfacing can then consist of a simple water blast of the remaining wearing surface and reapplication of one or two coats of the epoxy resin and broadcast overlay aggregate. Otherwise the wearing surface will need to be reapplied in accordance with the original procedures. The bottom coat of epoxy resin can be tinted with a different color than the top coats to alert maintenance that wear of the wearing surface has reached the bottom coat.

<u>Friction Stir Welded Joints</u>: Friction-stir welding (FSW), developed by The Welding Institute in 1991, is a solid-state, hot-shear joining process, where a rotating tool moves along the joint between butting surfaces of two rigidly clamped plates or extruded profiles. The tool includes a shoulder positioned above and in direct contact with the surface of the plates and a smaller threaded pin positioned within the depth of the plate. The tool shoulder makes firm contact with the top of the plates and generates heat by friction at the shoulder surface and, to a lesser degree, at the pin surface. Softening of material from the heat and rapid rotation of the tool produces plastic deformation and flow of the material. The plasticized material is transported from the front of the tool to the trailing edge as the tool advances. The material recrystallizes and forges into a solid joint as the material cools. (See Figure 12)

[3] Zhang (1999), *An Evaluation of the Durability of Polymer Concrete Bonds to Aluminum Bridge Decks*

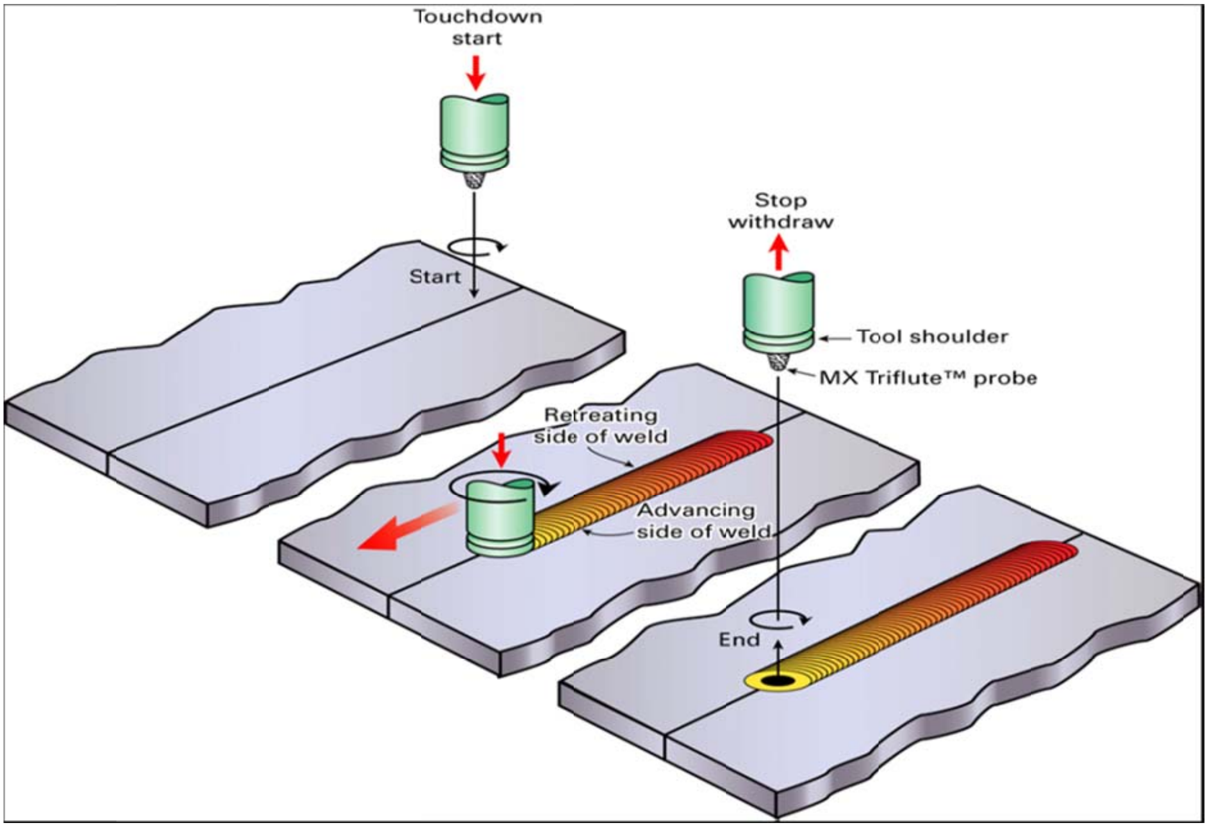


Figure 122 - Friction Stir Welding Process

FSW involves complex thermo-mechanical processes where varying deformation and temperature yields varying recrystallization of the plasticized material with different resulting microstructures within the limits of the joint. Temperatures of the plasticized material are below (typically 0.7 to 0.9) the melting point of the material. The combination of translation and rapid rotation of the tool yields a slightly asymmetric weld profile about the joint axis. (See Figure 13) The FSW joint consists of several distinct zones including:

- Weld Nugget (Fine-grained, Homogeneous, Fully Plasticized and Recrystallized Microstructure)
- Thermo-mechanically Affected Zone (TMAZ) (Variable-grained, Inhomogeneous, Partially Plasticized and Recrystallized Microstructure)
- Heat-affected Zone (HAZ) (Non-plasticized, Softened Normal-grained Microstructure)
- Unaffected Material

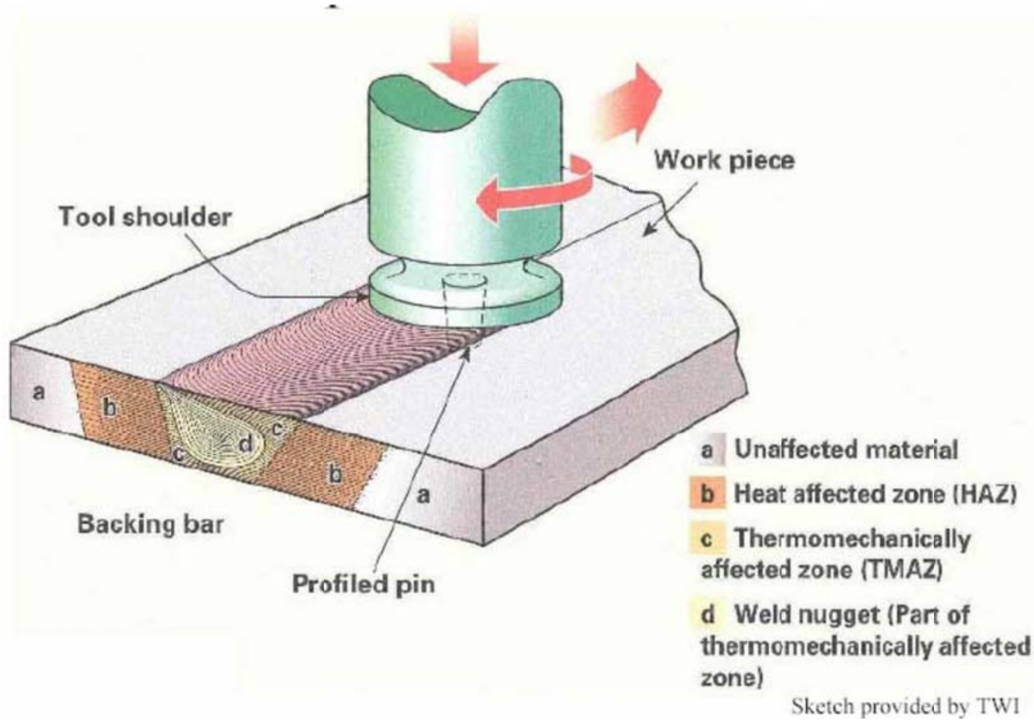


Figure 13 – Friction Stir Welded Joint

The FSW process for the aluminum orthotropic deck is automated using a machine developed specifically for FSW welding of aluminum deck systems. The FSW is single-sided with use of built-in backing seat and vertical web that are integral to the extrusions and that resist the applied vertical clamping force. The welds for the top and bottom plates are performed simultaneously so that the forces are self-reacting. (See Figures 14 thru 17)

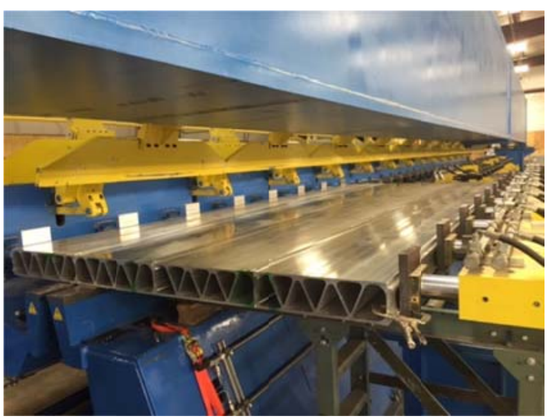


Figure 14 – Friction Stir Welding Machine

Figure 15 –Friction Stir Welding Machine

Figure 16 – Top Plate Friction Stir Welding

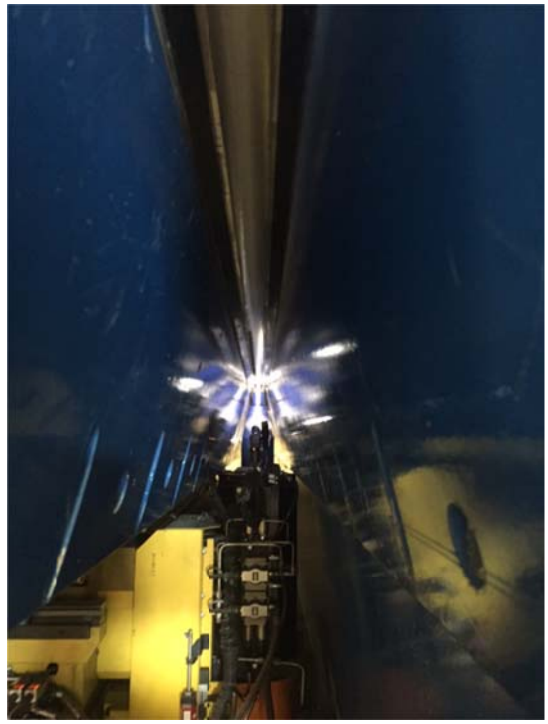


Figure 17 – Bottom Plate Friction Stir Welding

The single-sided friction stir welding with equal top and bottom weld sizes yields significantly lower weld distortion and flatter deck panels than the previous two-sided friction stir welding and unequal top and bottom weld sizes. It also permits faster and more efficient welding, which reduces fabrication costs.

Fatigue crack development is often associated with material and weld defects. Although FSW can produce welded joints of a much higher quality than that of metal inert gas (MIG) welding, FSW can still yield flaws that may contribute to the development of fatigue cracks including:

- Voids
- Lack of fusion
- Lack of penetration
- Faying surface defects
- Presence of entrapped oxides that can affect fusion
- Tooling marks.

The quality of the friction stir welds is dependent on a number of factors that are influenced by the following:

- Tooling including:
    - Width of the tool shoulder,
    - Height, depth and thread configuration of the tool pin (probe)
    - Adequate support of the tool(s) that prevents lift-off during testing.
- Friction stir welding processes including:
    - Welding speed (rotational and advancement),
    - Tool inclination angle

    - Welding pre-load force.
- Adequate clamping of the profiles that preloads and prevents separation.

Significant advancements and experience in FSW processes has greatly reduced the potential for weld defects. The above factors are controlled by the panel supplier. Although FSW is a different process than traditional aluminum welding, quality control of the FSW joint is performed using similar methodology to that of MIG welded joints including radiographic and/or ultrasonic NDT, coupon sampling and testing, and hardness tests. Welding and Weld Quality Control requirements for FSW joints are specified in AWS D1.2, Structural Welding Code – Aluminum (2014 Edition) and shall be implemented by the deck panel fabricator as follows:

- Aluminum deck will be considered as cyclically loaded, tubular structures in establishing requirements for welding and weld inspection.
- Joining of extrusions will only be made with friction stir welds and friction stir welds shall be considered complete joint penetration groove welds.
- A Procedure Qualification Records (PQR) shall be prepared and submitted to the Engineer for approval for each Weld Procedure Specification (WPS) including those for weld repairs.
- Welding shall only be performed by qualified welders. A Welder Performance Qualification Record (WPQR) shall be prepared and submitted for approval for each welder, welding operator, and tack welder performing the welding or weld repairs.
- Records shall be maintained for each weld including the panel identification, WPS used, date welded, welder, weld location, identified defects, and weld repairs.
- All welds shall be inspected by AWS certified weld inspectors.
- All welds shall be visually inspected prior to grinding.
- A tension test, bend test and macroetch test shall be performed on one weld tab for both the top and bottom plate of each panel. If the test results do not equal or exceed the acceptance criteria, the full length of the weld shall be inspected using ultrasonic inspection (UT).
- All welds shall be inspected using UT at an initial frequency of 10% of the length of the welds. If welding does not pass the acceptance criteria for cyclically loaded, tubular structures, the full length of the failed weld shall inspected and supplemental bend tests and macroetch tests shall be performed on weld tabs corresponding to the failed weld.
- Records shall be maintained for weld inspection and testing including panel identification, inspection date, inspector, method, acceptance criteria, results, and disposition.

<u>Panel Fabrication Requirements and Dimensional Tolerances</u>: Panels shall be required to meet the following requirements and dimensional tolerances after fabrication. Panel fabricator shall be responsible for performing required inspections and measurements, documenting, and making corrective actions:

- All exposed edges, except at top surface, shall be ground smooth to a 1/4” radius.
- Scratches and dents that exceed the limits in AWS D1.2, Table 5.3, shall be removed by grinding smooth. Repaired areas shall be inspected using dye-penetrant testing (PT). Parts that reveal cracks after PT and/or do not meet the dimensional requirements shall not be used.

- Dimensional tolerances relative to nominal value:

PARAMETER	TOLERANCE
Length	+/- 1/4”
Width	-1/4”, +1/2”
Squareness (Diagonal Variation)	+/- 1/4”
Flatness	1/2"
Edge Straightness	1/4"

- Measurement shall be performed using calibrated tools (e.g. steel tape, chains, straight edges, and machinist scales) accurate to at least 1/32” (0.03”). Measured values shall be reported to the nearest 1/16” (0.06”). Nominal dimensions shall be considered at baseline temperature of 70 deg F. Where temperatures at the time of measurement vary from 70 deg F, measured values shall be adjusted accounting for the difference in temperature. Records shall be maintained for dimensional tolerances including panel identification, measurement date, temperature, inspectors, tools, tool accuracy, nominal values, measured values, and difference between measured and nominal. Wearing surface shall not be applied until fabricated panels have been recorded, submitted to and approved by the Engineer.

Deck Panel to Stringer Assembly: The deck panel to stringer assembly shall be shop fabricated and assembled as follows after the fabricated deck panels have been delivered to a steel fabricator with the wearing surface already installed: (See Figures 18 thru 23):

- Stringers shall be fabricated in accordance with the latest editions of the Florida Department of Transportation Standard Specifications for Road and Bridge Construction, corresponding Supplemental Specifications and referenced AWS D1.5 Bridge Welding Code. Stringers shall receive specified protective coatings previously certified by RCSC as meeting requirements for ASTM A325 Class ‘B’. Bottom of deck shall receive abrasion blast finish at faying surface.
- Alignment of the deck panels and stringers shall be established in a floor layout for each floorbeam bay prior to drilling bolt holes. Floor layout shall be of sufficient accuracy to establish alignment at the deck panel ends and expansion joints within specified tolerances. Layout may be performed with the panels inverted. Care shall be taken to protect the wearing surface from damage. Hole locations in the stringer flange shall be established at assembly such that corresponding holes in the deck will be accurately located within the deck panel voids. The stringers and deck panels shall be match marked during initial alignment so that the alignment can be re-established at any stage during the operations.
- Bolt holes in the stringer flange shall be drilled using a portable magnetic drill and made with the stringers removed from the layout assembly and prior to drilling corresponding holes in the deck.
- Before drilling holes in the deck panels, steel stringers and deck panels shall be brought into matching temperatures, within 5 deg F, using heating blankets or other approves methods. Deck panels shall not be heated to a temperature greater than 150 deg F. Holes in the deck shall be made using the holes in the stringer flange as a template. Stringers shall be adequately

secured to the panels by way of blocking and/or clamping to prevent relative movement during drilling operations.

- Deck panels and stringers shall be bolted together after all holes are drilled. Bolt installation and tensioning shall be performed one stringer at a time starting either at one end of the panel. The first panel shall be fully bolted to the stringer, starting at the deck panel voids near the center of the panel and working in progressive sequence on each side of the center until the connection for the panel is complete. The second panel shall then be bolted to the stringer in a similar fashion. This process shall be followed until all stringers are bolted to the deck panels.

Figure 18 – Transferring Holes to Deck Panel

Figure 19 – Sand Blasting Panel Bottom

Figure 20 – Bolt Installation

Figure 21 – TC Bolt Tightening

Figure 22 – Completed Deck Unit

Figure 23 – Completed Deck Unit Shipping

2.0 PRELIMINARY STRUCURAL ANALYSIS AND EVALUATION

Preliminary finite element analysis (FEA) and manual calculations have been performed of the deck system for the proposed test panel size, support configuration, and each of the proposed loading scenarios. The purpose of this analysis was to:

- Analyze and evaluate adequacy of the proposed deck panel design (i.e. proposed deck panel extrusion profile dimensions and span capability) for AASHTO LRFD design loading, stress limits including strength, fatigue stress, and deflection limits,
- Analyze and evaluate deck panel load distribution, corresponding shear lag effects, and combined System 2 and 3 stress effects, and compare with simple closed-form equations, to avoid need for FEA for each deck design,
- Verify loading configuration that produces maximum effects,
- Identify locations for strain gauge placement required for meaningful comparison of analytical and experimental results,
- Determine the number and pitch of fasteners required to resist slip at Service II Live Loading.

Aluminum orthotropic decks have traditionally been evaluated for a combination of stresses using orthotropic plate theory as follows:

<u>System 1 Stresses</u>: Longitudinal compression stresses in the deck panels, introduced as a result of loading of the simply supported longitudinal support members (stringers) and corresponding deformations with the deck panels acting compositely with the supporting members. {NOTE: For the proposed Phase 1 (Component Testing), the support conditions will not generate System 1 Stresses. System 1 Stresses will be included in subsequent testing.]

<u>System 2 Stresses</u>: Transverse flexural compression or tension stresses and corresponding deformations in the deck panels introduced as a result of loading of the deck panels between the longitudinal support members (stringers). Panels experience positive and negative flexural stresses at different locations and loading conditions due to continuity of the deck over intermediate supports. Because stresses are distributed throughout the deck by way of the inclined webs, System 2 Stresses in the top and bottom plates vary due to shear lag effects with slightly higher stresses at the juncture of the top and bottom plates with the inclined webs and slightly lower stresses between the inclined webs.

<u>System 3 Stresses</u>: Localized flexural compression or tension stresses and corresponding deformations in the deck top plate and adjacent inclined webs introduced from wheel patch loads acting on the deck top plate. Because the top plate, bottom plate, and inclined webs are integral, the extrusions experience frame action.

[NOTE: System 2 Stresses are normal to System 1 and System 3 Stresses and thus are not directly additive. However, because of bi-axial states of stress, loading for System 3 Stresses also includes corresponding stresses that are parallel and additive to System 2 Stresses. System 1 and System 3 Stresses are parallel and thus are directly additive.]

2.1 SERVICE AND STRENGTH LIMIT STATES

Loading Magnitudes: The deck panel performance was analyzed and evaluated in accordance with AASHTO LRFD Articles 7.5.1 thru 7.5.3. Static loading was configured and applied in accordance with AASHTO LRFD Live Load (LL) and Dynamic Load Allowance (IM) in Articles 3.6.1.1, 3.6.1.2 and 3.6.2. Loading magnitudes are based on Load Combinations and Load Factors, Υ_{LL}, in Article 3.4.1 at the Service I (Deflections), Service II (Slip-Critical Connections), Strength I and Strength II Limit States. [NOTE: Strength II Limit State evaluates the deck for the Florida FL-120 Overload Permit Vehicle.]

The deck system was evaluated for AASHTO LRFD HL-93 Design Truck and Design Tandem, and Florida FL-120 Overload Permit Truck wheel loads. Based on preliminary finite element analysis, it appears that the AASHTO LRFD HL-93 Design Truck governs over the Design Tandem, although the difference in magnitude of the stresses is relatively small.

By inspection, a single lane of traffic governs over multiple lanes when considering AASHTO LRFD Multiple Presence Factors, *m*. For example, in evaluating System 2 Stresses, a single-lane of traffic (Multiple Presence Factor, *m* = 1.20) with two wheel lines spaced at 6'-0" on center produces greater negative moment intensity than two lanes of traffic (Multiple Presence Factor, *m* = 1.00) with adjacent wheel lines from different lanes spaced at 4'-0" on center. For similar reasons, a single-lane of traffic (i.e. single wheel line) produces greater positive moment intensity than two lanes of traffic.

Wheel loads were applied in the configurations shown in the Test Set-up Drawings and as described below:

1. Apply wheel loads to deck top surface as 20" (transverse) x 10" (longitudinal) patch load using a neoprene pad to uniformly distribute load.

2. Apply wheel loads at the loading levels below.

TABLE 1 - STATIC WHEEL PATCH LOADS (kips) (1)

AASHTO LRFD LOAD CASE	LOAD FACTOR, Υ_{LL}	HL-93 DESIGN TRUCK	HL-93 DESIGN TANDEM
SERVICE I	1.00	25.54 (2)	19.95 (3)
SERVICE II	1.30	33.20 (2)	25.94 (3)
STRENGTH I	1.75	44.69 (2)	34.91 (3)
STRENGTH II	1.35	57.46 (4)	N/A

TABLE FOOTNOTES:

(1) Wheel patch loads calculated as follows: $\Upsilon_{LL}\,Q(1 + IM)m$

(2) AASHTO LRFD HL-93 Design Truck 32-kip Rear Axle (Q=16-kip Wheel), magnified for Dynamic Load Allowance (IM=0.33) and Single-Lane Multi-Presence Factor (m=1.20).

(3) AASHTO LRFD HL-93 Design Tandem 25-kip Axle (Q=12.5-kip Wheel), magnified for Dynamic Load Allowance (IM=0.33) and Single-Lane Multiple Presence Factor (m=1.20).

(4) FDOT FL-120 Permit Truck (HL-93 Truck Magnified x 1.67) 53.33-kip Rear Axle (Q=26.67-kip Wheel), magnified for Dynamic Load Allowance (IM=0.33) and Single-Lane Multi-Presence Factor (m=1.20).

System 2 Static Loading Configurations and Analysis: Analysis for static loading is generally as follows:

- *Analysis Approach*: System 2 Stresses and Deflections were computed using three-dimensional plate and shell models using LUSAS computer software. Loading was based on 10 kip unit wheel patch loads and the stresses and deflections magnified based on the wheel patch loads in Table 1. Plots of the System 2 Stresses and Deflections were made to summarize the results. (See Figure 24)

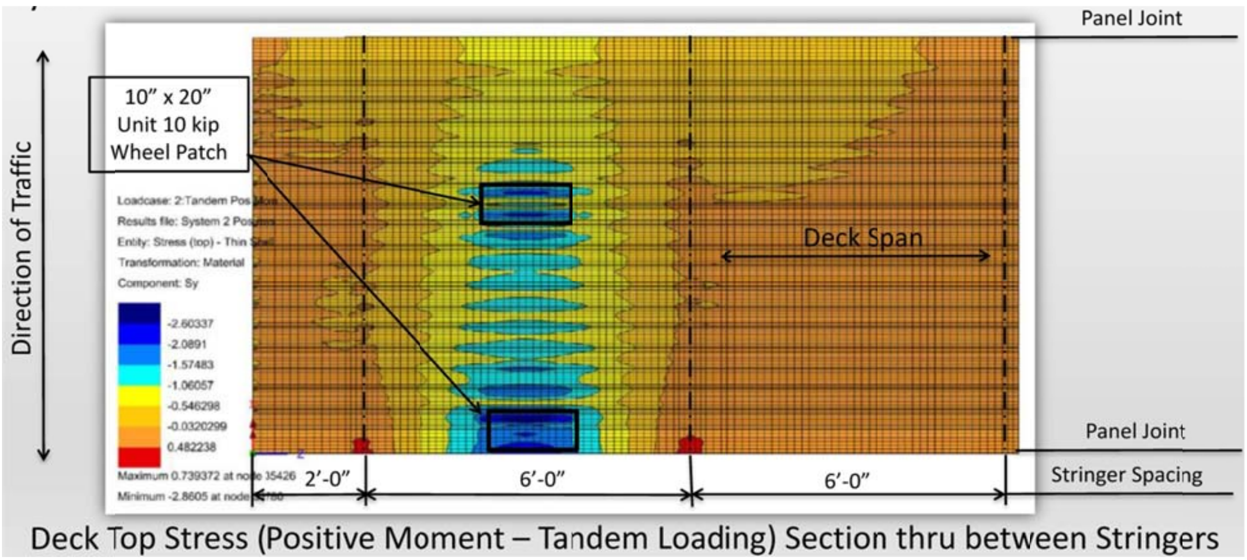


Figure 24 - FEA (3D Plate and Shell Model) Typical Stress Contour

- *Positive Flexure*: The deck experiences maximum positive flexure (tension in the bottom of the deck and compression in the top of the deck) between the stringers with the primary stresses oriented perpendicular to the direction of the moving load. A single wheel line located mid-distance between the supports, with one wheel patch always located adjacent to the panel transverse edge, produces maximum stresses and deflection for positive flexure. A simple-span configuration produced conservative results. For the HL-93 Design Truck or FL-120 Permit Truck, a single wheel patch is applied. For the HL-93 Design Tandem, two wheel patches spaced longitudinally at 4'-0" on center are applied. Based on FEA, the calculated maximum stresses and deflections for each of the loading scenarios are as follows:

TABLE 2 – SYSTEM 2 POSITIVE FLEXURE MAXIMUM STRESSES AND DEFLECTIONS

Limit State	Loading	Max. Stress (ksi)		Max. Deflection (in)
		Tension (Bottom)	Compression (Top)	
Service I	HL-93 Design Truck	6.3	6.7	0.090
	HL-93 Design Tandem	5.6	5.7	0.082
Service II	HL-93 Design Truck	8.2	8.7	0.117
	HL-93 Design Tandem	7.3	7.4	0.107

Strength I	HL-93 Design Truck	11.1	11.7	0.158
	HL-93 Design Tandem	9.7	10.0	0.144
Strength II	FL-120 Permit Truck	14.3	15.0	0.203
Limits	$\Phi_f F_{nb}$ =	26.4	26.3	0.090

Stress and deflection contours for Positive Flexure are shown below (See Figures 25 thru 30):

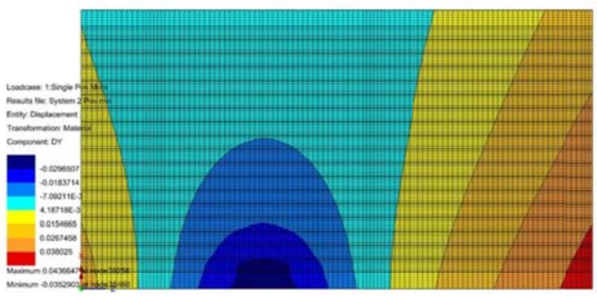


Figure 25 – Deflection Contour – Truck Loading

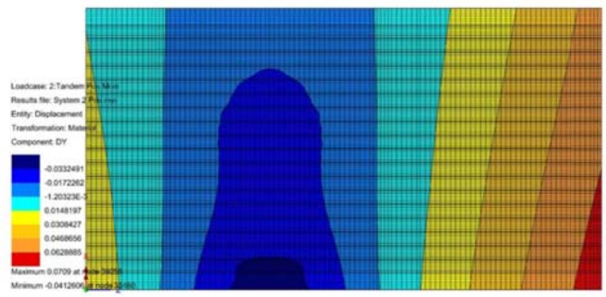


Figure 26 – Deflection Contour – Tandem Loading

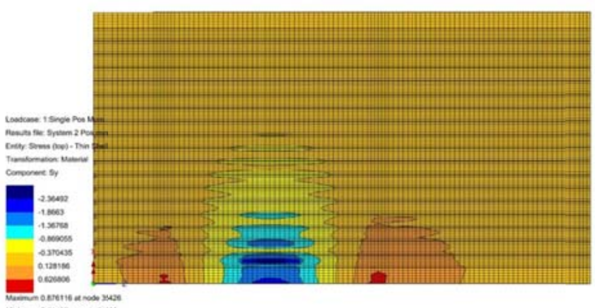


Figure 27 – Top Plate Stress Contour – Truck Loading

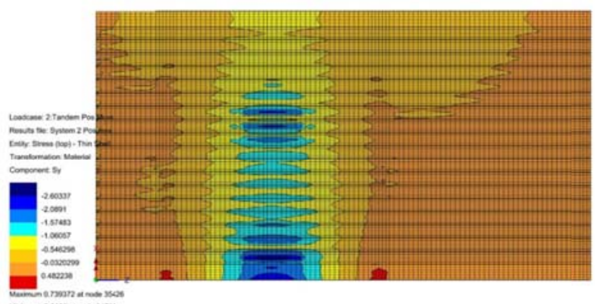


Figure 28 – Top Plate Stress Contour – Tandem Loading

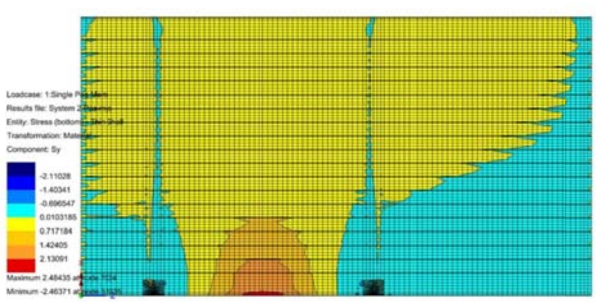


Figure 29 – Bottom Plate Stress Contour – Truck Loading

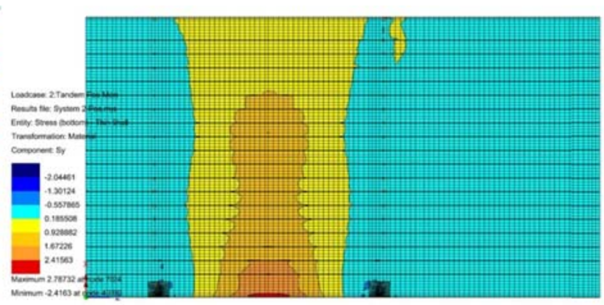


Figure 30 – Bottom Plate Stress Contour – Tandem Loading

- *Negative Flexure*: The deck experiences maximum negative flexure (tension in the top of the deck and compression in the bottom of the deck) over the intermediate support (intermediate stringer) with the primary stresses oriented perpendicular to the direction of the moving load. A pair of wheel lines spaced at 6'-0" on center transversely and centered over the intermediate support, with one wheel patch always located adjacent to the panel transverse edge, produces maximum stress for negative flexure in a two-span continuous support configuration. For the HL-93 Design Truck or FL-120 Permit Truck, a single wheel patch per wheel line is applied. For the HL-93 Design Tandem, two wheel patches per wheel line spaced at 4'-0" on center longitudinally are applied. The calculated maximum stresses and deflections for each of the loading scenarios are as follows:

TABLE 3 –SYSTEM 2 NEGATIVE FLEXURE MAXIMUM STRESSES AND DEFLECTIONS				
Limit State	Loading	Max. Stress (ksi)		Max. Deflection (in)
		Tension (Top)	Compression (Bottom)	
Service I	HL-93 Design Truck	6.3	8.7	0.069
	HL-93 Design Tandem	5.0	6.8	0.056
Service II	HL-93 Design Truck	8.2	11.3	0.090
	HL-93 Design Tandem	6.5	8.8	0.073
Strength I	HL-93 Design Truck	11.0	15.1	0.121
	HL-93 Design Tandem	8.7	11.9	0.097
Strength II	FL-120 Permit Truck	14.2	19.5	0.156
Limits	$\Phi_f F_{nb}$ =	26.2	26.4	0.090

Stress and deflection contours for Negative Flexure are shown below (See Figures 31 thru 36):

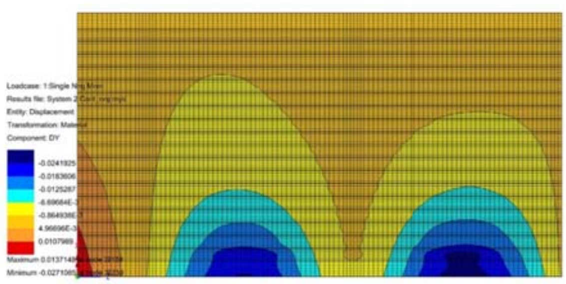


Figure 31 – Deflection Contour – Truck Loading

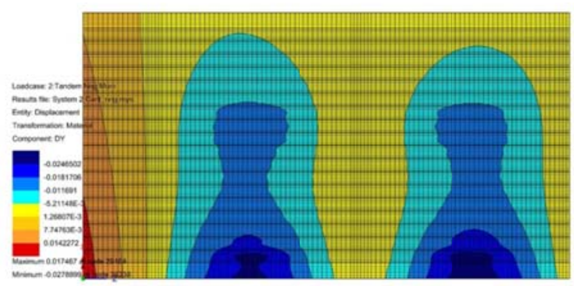


Figure 32 – Deflection Contour – Tandem Loading

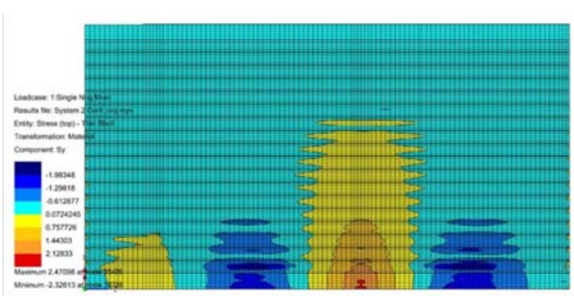


Figure 33 – Top Plate Stress Contour – Truck Loading

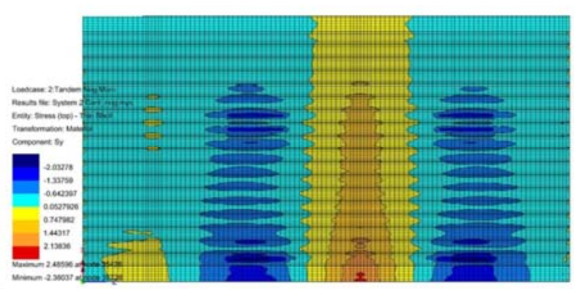
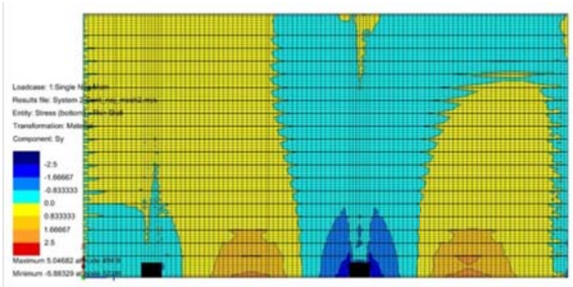


Figure 34 – Top Plate Stress Contour – Tandem Loading



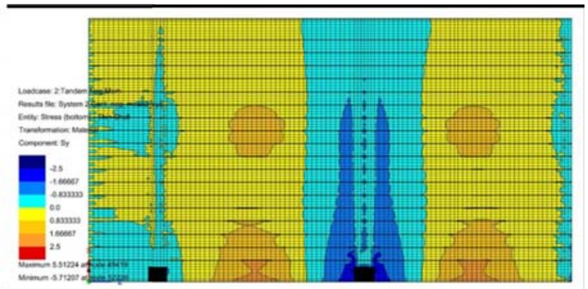
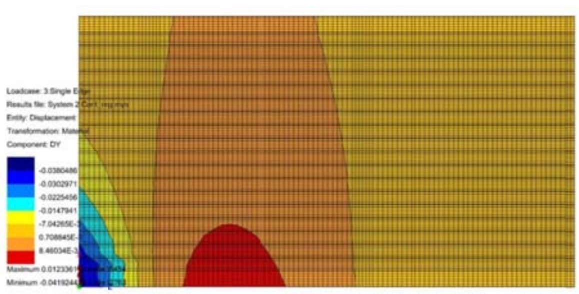


Figure 35 – Bottom Plate Stress Contour – Truck Loading Figure 36 – Bottom Plate Stress Contour – Tandem Loading

- *Cantilever Negative Flexure*: The deck experiences maximum negative flexure (tension in the top of the deck and compression in the bottom of the deck) over the exterior support (exterior stringer) with the primary stresses oriented perpendicular to the direction of the moving load. A single wheel line on the cantilever, located 1'-0" transversely from the center of the exterior support, with one wheel patch always located adjacent to the panel transverse edge, produces maximum stresses and deflections for negative flexure in the cantilever support configuration. For the HL-93 Design Truck or FL-120 Permit Truck, a single wheel patch per wheel line is applied. For the HL-93 Design Tandem, two wheel patches per wheel line spaced at 4'-0" on center longitudinally are applied. The calculated maximum stresses and deflections for each of the loading scenarios are as follows:

TABLE 4 – SYSTEM 2 CANTILEVER NEGATIVE FLEXURE MAXIMUM STRESSES AND DEFLECTIONS				
Limit State	Loading	Max. Stress (ksi)		Max. Deflection (in)
		Tension (Top)	Compression (Bottom)	
Service I	HL-93 Design Truck	5.7	8.1	0.107
	HL-93 Design Tandem	4.6	6.8	0.097
Service II	HL-93 Design Truck	7.4	10.5	0.139
	HL-93 Design Tandem	6.0	8.8	0.126
Strength I	HL-93 Design Truck	10.0	14.3	0.187
	HL-93 Design Tandem	8.1	11.9	0.169
Strength II	FL-120 Permit Truck	12.9	18.3	0.241
Limits	$\Phi_f F_{nb}$ =	26.2	26.4	0.090

Stress and deflection contours for Cantilever Negative Flexure are shown below (See Figures 37 thru 42):



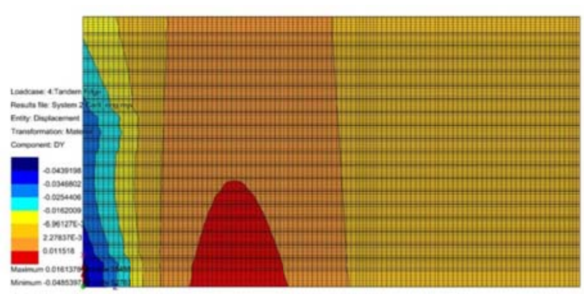


Figure 37 – Deflection Contour – Truck Loading Figure 38 – Deflection Contour – Tandem Loading

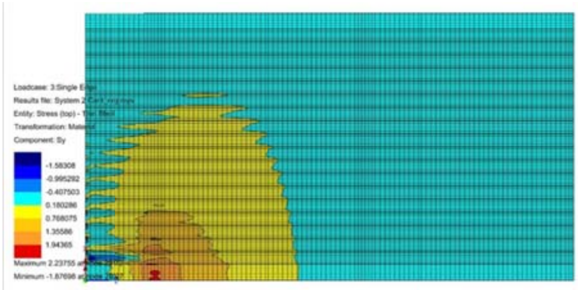


Figure 39 – Top Plate Stress Contour – Truck Loading

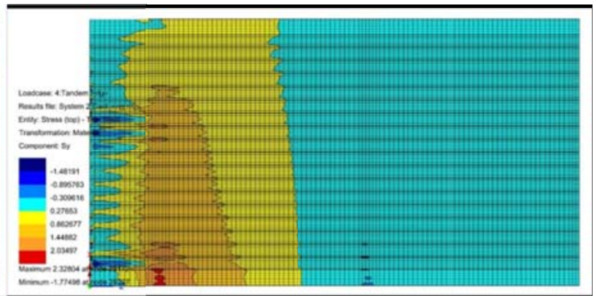


Figure 40 – Top Plate Stress Contour – Tandem Loading

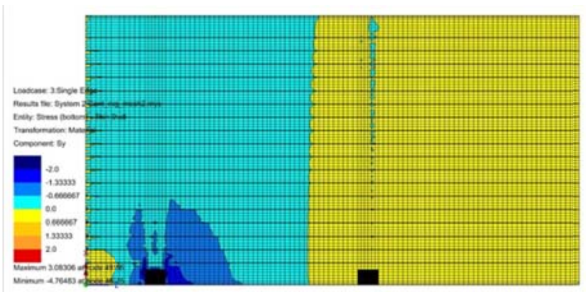


Figure 41 – Bottom Plate Stress Contour – Truck Loading

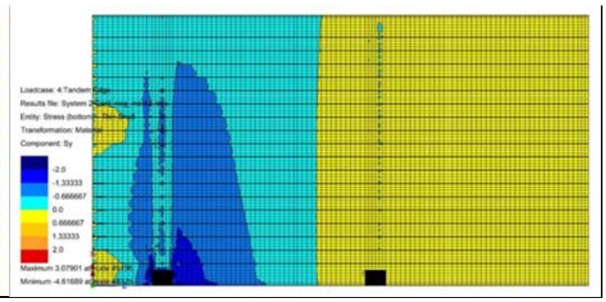


Figure 42 – Bottom Plate Stress Contour – Tandem Loading

- *Shear Lag Effects*: Because stresses are distributed throughout the deck by way of the inclined webs, System 2 Stresses in the top and bottom plates vary due to shear lag effects with slightly higher stresses at the juncture of the top and bottom plates with the inclined webs and slightly lower stresses between the inclined webs. The shear lag effects are exhibited in the stress contours as "ripples". The magnitude of the shear lag effects were evaluated by analyzing the variation in stress within the top and bottom plates. FEA System 2 Positive Flexure Stresses (Truck Loading) for the top and bottom plates (top, mid and bottom surfaces of the plates) were plotted along a longitudinal line mid-distance between the stringers (i.e. along the applied wheel line.) Trendlines for each of the stress lines were then established and plotted. A shear lag multiplier was applied to the trendlines and adjusted until the magnified trendlines generally enveloped the FEA stresses. (See Figures 43 and 44) The shear lag multiplier for the top plate is 1.20 and for the bottom plate is 1.09. It is conservatively recommended to use a shear lag multiplier of 1.20 for both the top and bottom plates. The FEA results already include the effects of shear lag and thus values do not need to be magnified. However, stresses computed using simple closed-form equations need to be magnified by the shear lag multipliers to yield accurate results.

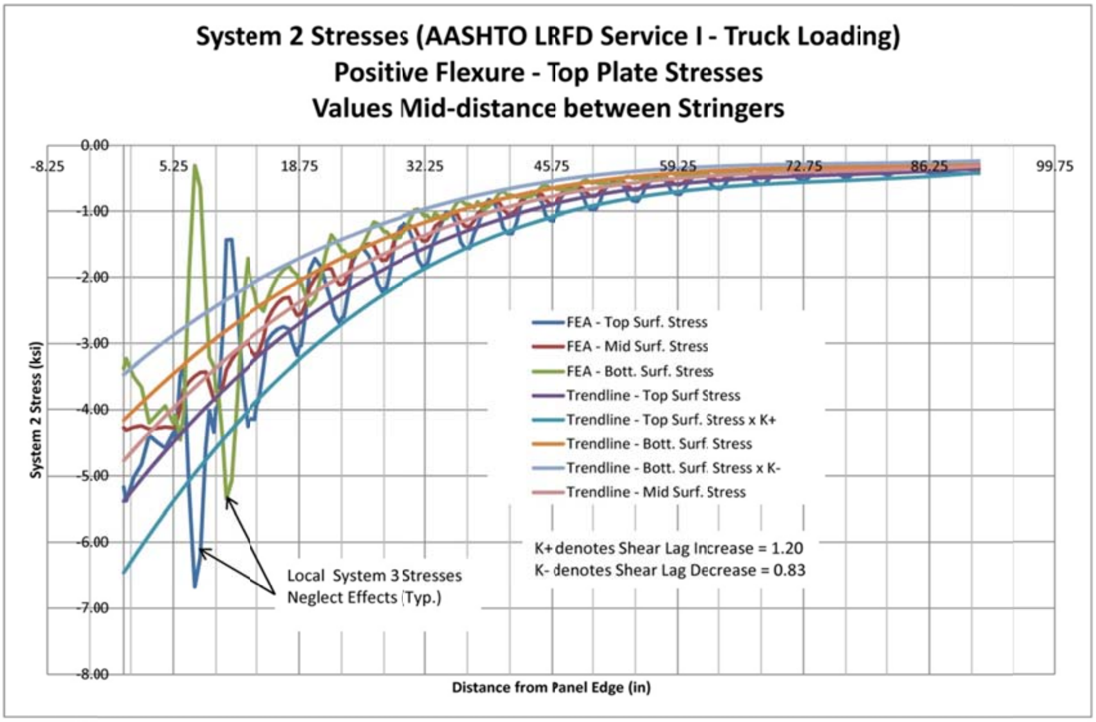


Figure 43 – Top Plate Longitudinal Stress Distribution showing Shear Lag Effects

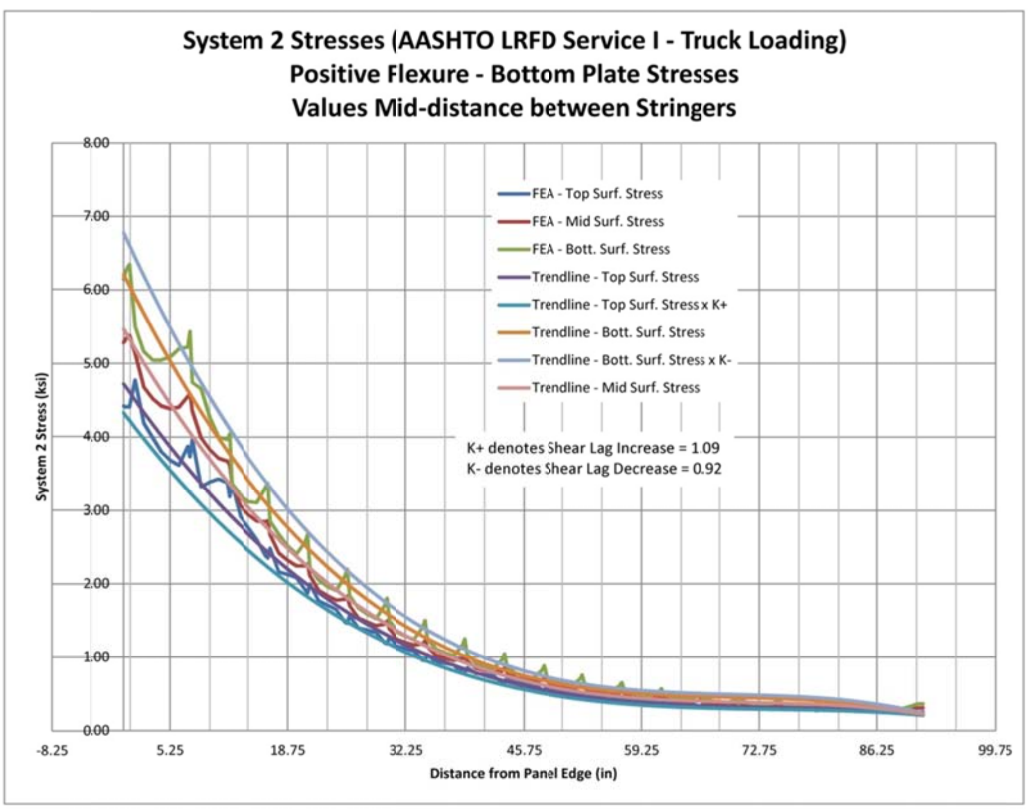


Figure 44 – Bottom Plate Longitudinal Stress Distribution showing Shear Lag Effects

- *Local System 3 Effects*: Wheel patch loading on the top plate introduces flexural stresses (System 3 Stresses) in the top plate. Although the direction of primary stress is in the longitudinal direction, there are corresponding secondary stresses in the transverse direction due to plate bending biaxial state of stress. These secondary stresses are directly additive to primary System 2 Stresses. For maximum positive flexure, the secondary wheel patch stresses correspond with the location of the maximum System 2 Stresses. In other governing maximum stress locations (e.g. maximum System 2 Negative Flexure Stress and maximum System 2 Cantilever Negative Flexure Stress), the location of the wheel patch secondary stresses does not correspond with the location of maximum System 2 Stress and thus the stresses need not be added. The magnitude of the secondary stresses that should be added to the maximum System 2 Positive Flexure Stresses are as follows:

TABLE 5 – SECONDARY STRESSES ADDED TO MAXIMUM SYSTEM 2 POSITIVE FLEXURE STRESSES	
Limit State	Top Plate Secondary Flexural Stress (ksi)
Service I	3.2 ksi
Service II	4.2 ksi
Strength I	5.6 ksi
Strength II	7.2 ksi

The FEA results already include the effects of local wheel patch loading and thus the secondary stress values need not be added to the System 2 Positive Flexure Stresses. However, when stresses are computed using simple closed-form equations, the above secondary stresses should be added to the maximum System 2 Positive Flexural Stresses to yield accurate results.

- *Simple Closed-Form Equations (Equivalent Strip Width)*: In order to avoid the need to perform FEA for each aluminum orthotropic deck design, simple closed-form equations for estimating the governing bending moment intensity in the aluminum orthotropic deck panels is preferable. The previous testing for the 8-inch aluminum orthotropic deck, performed by FHWA, Virginia Department of Transportation, and Virginia Tech, demonstrated that the live load moment distribution within the aluminum orthotropic deck panels closely matches that of a reinforced concrete deck. As such, the same simple closed-form equations (AASHTO LRFD Articles 4.6.2.1.3 and 4.6.2.1.4c) used to determine the equivalent strip width for reinforced concrete slab design can be used for aluminum orthotropic deck design. Comparison of FEA and simple-closed form equations confirms that similar results are obtained. The applicable equations for equivalent strip widths (in units of inches) are as follows:

Interior Strip Width (Positive Flexure):	$26.0 + 6.6S$
Interior Strip Width (Negative Flexure):	$48.0 + 3.0S$
Transverse Edge Width (Positive Flexure):	$X + 0.5*(26.0 + 6.6S)$
Transverse Edge Width (Negative Flexure):	$X + 0.5*(48.0 + 3.0S)$

Where: S denotes stringer spacing (ft)
X denotes distance from edge of deck panel to end of stringer (in)

2.2 FATIGUE LIMIT STATE

Fatigue Loading: The deck panel performance was analyzed and evaluated for fatigue in accordance with AASHTO LRFD Article 7.6.1. Fatigue loading was configured and applied in accordance with AASHTO LRFD Live Load (LL) and Fatigue Dynamic Load Allowance (IM) in Articles 3.6.1.4 and 3.6.2. (See Figure 45) Loading magnitudes are based on Load Combinations and Load Factors, Υ_{LL}, in Article 3.4.1 at the Fatigue I Limit State, which corresponds to infinite fatigue life.

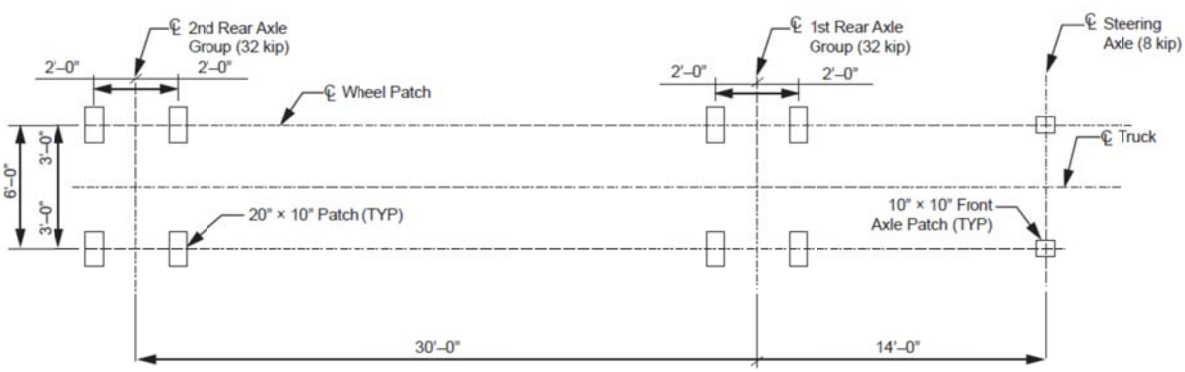


Figure 45 – Fatigue Truck

Each fatigue detail shall satisfy:

$\Upsilon\,(\Delta f) \leq (\Delta F)_N$ where (AASHTO LRFD Equation 7.6.1.2.2-1)

Υ denotes load factor, (Δf) denotes the force effect stress due to the passage of the design fatigue load, and $(\Delta F)_N$ denotes Nominal Fatigue Resistance (see below.)

In order to evaluate the performance of specific fatigue sensitive details, the details should be analyzed for a stress range equal to the design Nominal Fatigue Resistance, $(\Delta F)_N$ for a number of cycles considered equivalent to infinite fatigue life.

Wheel patch loads that correspond to the factored design fatigue load, $\Upsilon\,(\Delta f)$, are as follows:

$\Upsilon_{LL}\,Q(1 + IM)$ = 1.50 (8 kips) (1+0.15) = 13.8 kips where

Υ_{LL} denotes the Fatigue I Limit State load factor, Q denotes the fatigue truck wheel load and IM denotes the fatigue dynamic load allowance.

Fatigue Sensitive Details: The fatigue sensitive details for the aluminum orthotropic deck system are generally classified in accordance with AASHTO LRFD Article 7.6.1.2.3 with the following clarifications:

1. *Base Metal*: Category 'A' [NOTE: Applies to base metal throughout the deck panels loaded for System 1, 2 or 3.]

2. *Welded Joint (Stresses Normal to Weld Axis)*: Category 'C' [NOTE: This detail applies to the welded joint in the top plate subject to System 3 Stresses. This fatigue detail classification is consistent with AASHTO LRFD for complete joint penetration groove welded splices with primary stresses normal to the axis of the weld. The friction stir welding produces a smooth weld profile and surface condition similar to that produced by grinding of a weld. In previous laboratory tests for similar aluminum orthotropic decks, the testing verified that the welded joint detail provided fatigue resistance equal to or better than Category 'C'. With the previous aluminum orthotropic decks, the welded splices were made using metal inert gas (MIG) welding. MIG welding produces a larger heat affected zone, greater distortion and residual tensile stresses, and higher likelihood of weld defects that adversely affect fatigue resistance. Fatigue testing of friction stir welded joints of Alloy 6063-T6 material has demonstrated good fatigue resistance (equal or better than that of the base metal in some instances.) Although the friction stir welding is known to produce higher quality welded joints with greater fatigue resistance, the welded joint will be conservatively classified as Category 'C' for the purpose of this analysis.]
3. *Mechanically Fastened Connections*: Category 'C' to Category 'E' depending on the stress ratio. [NOTE: Fatigue of the base metal at the net section through the holes for the mechanical fasteners is not considered in the analysis. This portion of the deck will remain in compression under all fatigue loading scenarios and thus is not subject to tension or stress reversal.]
4. *Welded Joint (Stresses Parallel to Weld Axis)*: Category 'B' [NOTE: This detail applies to the welded joint in the top and bottom plate subject to System 2 Stresses. This fatigue detail classification is consistent with AASHTO LRFD for complete joint penetration groove welded splices with the stresses parallel to the axis of the weld.]

<u>Stress Range and Number of Fatigue Cycles</u>: The number of cycles, N, of stress range corresponding to infinite fatigue life for the AASHTO LRFD fatigue design loading is listed in the table below for each of the fatigue sensitive details. This relationship is based on the following equation:

$$(\Delta F)_N = \left(\frac{C_1}{N}\right)^{C_2} \geq \tfrac{1}{2}(\Delta F)_{TH} \text{ where} \qquad \text{(AASHTO LRFD Equation 7.6.1.2.4-1)}$$

$(\Delta F)_N$ denotes Nominal Fatigue Resistance, C_1 and C_2 denote Constants representing the x-intercept and slope, respectively, of the logarithmic S-N curves, N denotes number of cycles, and $(\Delta F)_{TH}$ denotes Constant Amplitude Fatigue Threshold for the specific Fatigue Detail. (See Figure 46)

[NOTE: As noted in AASHTO LRFD Commentary Article C7.6.1.2.4, the Design Nominal Fatigue Resistance, $(\Delta F)_N$ is considered to be one-half the Constant Amplitude Fatigue Threshold, $\tfrac{1}{2}(\Delta F)_{TH}$, in recognition that "the <u>maximum</u> stress range is assumed to be twice the live load stress range due to the passage of the factored <u>design</u> fatigue load." This provision recognizes that actual traffic produces variable amplitude loading and includes trucks of a wide range of gross vehicle weights and axle configurations. Although actual traffic produces variable amplitude loading, the fatigue design provisions are based on constant amplitude fatigue loading for simplicity. The design fatigue loading, load factors, dynamic load allowance, and nominal fatigue resistance have been established and calibrated to be equivalent to the actual variable amplitude traffic loading.]

TABLE 6 - INFINITE FATIGUE VARIABLES				
Variable	Category 'A'	Category 'B'	Category 'C'	AASHTO LRFD Ref.
$(\Delta F)_{TH}$	9.5 ksi	6.0 ksi	4.0 ksi	Table 7.6.1.2.4-3
Design $(\Delta F)_N$=½$(\Delta F)_{TH}$	4.75 ksi	3.00 ksi	2.0 ksi	Equation 7.6.1.2.4-1
C_1	100,000x10^8	520.0x10^8	36.0x10^8	Table 7.6.1.2.4-1
C_2	0.155	0.211	0.237	
N at $(\Delta F)_{TH}$	5x10^6	10x10^6	10x10^6	Equation 7.6.1.2.4-1
N at $(\Delta F)_N$=½$(\Delta F)_{TH}$	430x10^6	285x10^6	193x10^6	Equation 7.6.1.2.4-1
$(ADTT)_{SL}$ for 75 Yr. Life	7850	5205	3525	Equation 7.6.1.2.4-2

The Average Daily Truck Traffic for a single lane, $(ADTT)_{SL}$, that would produce the number of cycles of Fatigue Design Loading equivalent to infinite fatigue life for a 75-year service life are based on the following equation:

$N = (365)\ (75)\ n\ (ADTT)_{SL}$ where (AASHTO LRFD Equation 7.6.1.2.4-2)

n denotes the Load Cycles per Truck Passage from AASHTO LRFD Table 7.6.1.2.4-2.

The $(ADTT)_{SL}$ values listed above are very conservative for most bascule bridges in Florida (i.e. most bascule bridges in Florida experience traffic volumes and corresponding truck percentages far less than these values.

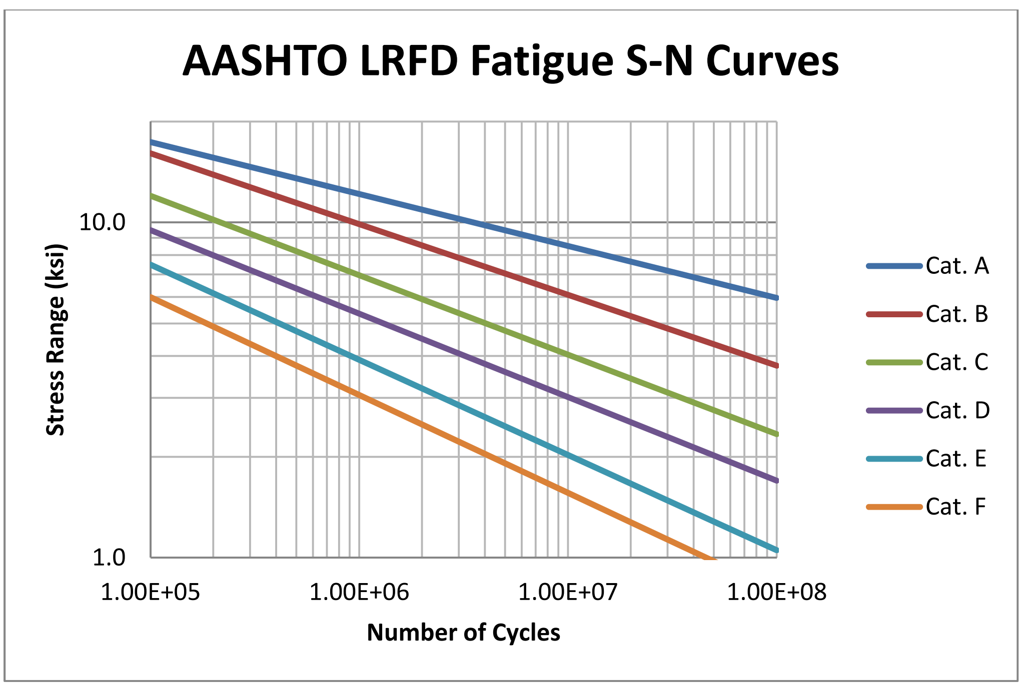


Figure 46 – AASHTO LRFD Fatigue S-N Curves

System 2 Fatigue Stresses: System 2 fatigue analyses was performed using the same three-dimensional plate and shell models used in the static Strength and Service Limit State Analysis. System 2 fatigue analyses evaluated the following:

- *Bottom of Panel in Positive Flexure*: The bottom of the deck panel between the supports (stringers) experiences tension stress range due to positive flexure with the primary stresses oriented perpendicular to the direction of the moving load. The base metal is considered Category 'A' while the FSW welded joints are considered Category 'B' with tension stress range parallel to the axis of the joint. A single wheel line located mid-distance between the supports with two wheel patch loads longitudinally spaced at 4'-0" on center produces the governing stress range, with the maximum stress range located at the edge of the test panel. Based on FEA, the calculated maximum tension stress range, produced by the moving wheel line for the factored fatigue design loading (13.8 kips per wheel patch), is approximately 3.85 ksi, which is slightly higher than one-half the Constant Amplitude Fatigue Threshold, $½(\Delta \mathrm{F})_{TH}$, of 3.00 ksi for Category 'B'. As such, this is a fatigue detail that will need additional scrutiny during laboratory testing. Given that FSW welded joints have shown to yield greater fatigue resistance than Metal Inert Gas (MIG) welding used to develop the fatigue thresholds, it is anticipated that this stress range will not be a concern.
- *Top of Panel in Negative Flexure*: The top of the deck panel over the intermediate supports (stringers) experiences tension stress range due to negative flexure with the primary stresses oriented perpendicular to the direction of the moving load. The base metal is considered Category 'A' while the FSW welded joints are considered Category 'B' with tension stress range parallel to the axis of the joint. A pair of wheel lines transversely spaced at 6'-0" on center and centered over the intermediate support with two wheel patch loads each longitudinally spaced at 4'-0" on center produces the governing stress range, with the maximum stress range located at the edge of the test panel. Based on FEA, the calculated maximum tension stress range, produced by the moving wheel line for the factored fatigue design loading (13.8 kips per wheel patch), is approximately 3.44 ksi, which is slightly higher than one-half the Constant Amplitude Fatigue Threshold, $½(\Delta \mathrm{F})_{TH}$, of 3.00 ksi for Category 'B'. As such, this is a fatigue detail that will need additional scrutiny during laboratory testing. Given that FSW welded joints have shown to yield greater fatigue resistance than Metal Inert Gas (MIG) welding used to develop the fatigue thresholds, it is anticipated that this stress range will not be a concern.
- *Top of Panel in Cantilever Negative Flexure*: The top of the deck panel over the outboard support (stringer) adjacent to the deck cantilever experiences tension stress range due to negative flexure with the primary stresses oriented perpendicular to the direction of the moving load. The base metal is considered Category 'A' while the FSW welded joints are considered Category 'B' with tension stress range parallel to the axis of the joint. A single wheel line transversely spaced at 1'-0" outboard the support with two wheel patch loads each longitudinally spaced at 4'-0" on center produces the governing stress range, with the maximum stress range located at the edge of the test panel. Based on FEA, the calculated maximum tension stress range produced by the moving wheel line for the fatigue design loading (13.8 kips per wheel patch) is approximately 3.22 ksi, which is slightly higher than one-half the Constant

Amplitude Fatigue Threshold, ½$(\Delta F)_{TH}$, of 3.00 ksi for Category 'B'. As such, this is a fatigue detail that will need additional scrutiny during laboratory testing. Given that FSW welded joints have shown to yield greater fatigue resistance than Metal Inert Gas (MIG) welding used to develop the fatigue thresholds, it is anticipated that this stress range will not be a concern.

System 3 Fatigue Stress Loading: System 3 fatigue analyses was performed using a three-dimensional solid element model using the LUSAS software and included moving fatigue loading, thermal stress range, and braking loads. (See Figures 47 thru 49) System 3 fatigue analyses evaluated the following:

- *Base Metal in Top Plate between Inclined Webs*: The relatively thin (0.25" minimum) top plate between the inclined webs experiences stress reversal from the moving wheel loads. A single wheel patch load produces the governing stress range. Based on FEA, the calculated maximum stress range, produced by the moving wheel patch for the fatigue design loading (13.8 kips), is approximately 4.25 ksi, which is less than one-half the Constant Amplitude Fatigue Threshold, ½$(\Delta F)_{TH}$, of 4.75 ksi for Category 'A'. As such fatigue is not anticipated to be a concern with this fatigue detail.
- *Base Metal Immediately Adjacent to FSW Joint*: The FSW joint between the extrusion profiles experiences stress reversal from the moving wheel loads. A single wheel patch load produces the governing stress range. The tooling marks in the top of the deck plate from the FSW operations are considered to produce a potential fatigue sensitive detail. The edge of the tooling marks is located at a distance equal to the radius of the FSW tool shoulder, which is 0.5" (i.e. half of the 1" diameter shoulder.) Based on previous research, the HAZ for the FSW joint generally extends approximately 0.75" from the center of the joint with maximum softening (i.e. loss in base metal strength) occurring at approximately 0.5" from the center of the joint. The effects of the tooling marks are anticipated to govern the formation of fatigue cracking of the FSW joint and the softening of the base material generally does not have a significant effect on the fatigue resistance of the base metal. Conservatively, the FSW joint will be considered as Category 'C' for a distance of 0.5" from the center of the welded joint. Based on FEA, the calculated maximum stress range at 0.5" from the center of the FSW joint, produced by the moving wheel patch for the fatigue design loading (13.8 kips), is less than 0.63 ksi, which is less than one-half the Constant Amplitude Fatigue Threshold, ½$(\Delta F)_{TH}$, of 2.00 ksi for Category 'C'. As such fatigue is not anticipated to be a concern with this fatigue detail.
- *Base Metal in Inclined Webs*: The relatively thin (0.188" minimum) inclined webs, which are integral with the top plate, experience stress reversal from the moving wheel loads. A single wheel patch load produces the governing stress range. Based on FEA, the calculated maximum stress range produced by the moving wheel patch for the fatigue design loading (13.8 kips) is less than 3.25 ksi, which is less than one-half the Constant Amplitude Fatigue Threshold, ½$(\Delta F)_{TH}$, of 4.75 ksi for Category 'A' and lower than the stress range in the top plate produced from the same loading. As such, the inclined webs are not anticipated to control the fatigue design of the deck and not anticipated to be a concern with this fatigue detail.
- *Reentrant Corner at FSW Joint Seats*: The reentrant corner at the FSW joint seats introduces stress raisers that could be fatigue concern. This fatigue detail is not classified by AASHTO. A

single wheel patch load produces the governing stress range. Based on FEA, the calculated maximum stress range at the reentrant corner, produced by the moving wheel patch for the fatigue design loading (13.8 kips), is between 0.625 ksi and 1.25 ksi. The low stress range at the reentrant corner is limited by several factors. First, the FSW joint is significantly thicker than the surrounding top plates and vertical web members and contains a large fillet. Based on relative stiffness principals, the thin top plate and vertical web greatly relieves the stress in the thick welded joint. Second, longitudinal deformations from thermal and live load braking forces are resisted by the significant stiffness of the repeating inclined web patterns. Third, the FSW joint is located in a bay without inclined web members. Some of the macroetch testing of the trial FSW joints revealed lack of fusion through the horizontal leg of the "L" shaped joint and the presence of a thin horizontal feature along the joint seat and emanating from the reentrant corner. Because one-half of the FSW joint is solid, as a fully integral part of the extrusion, the risk is low that the horizontal feature will propagate into the solid half of the joint. Despite the above considerations, this is a fatigue detail that will need additional scrutiny during laboratory testing. Further discussion with HF Webster, the fabricator that performed the trial FSW, is warranted to determine if adjustments can be made to the FSW to achieve consistent fusion of the horizontal leg of the welded joint.

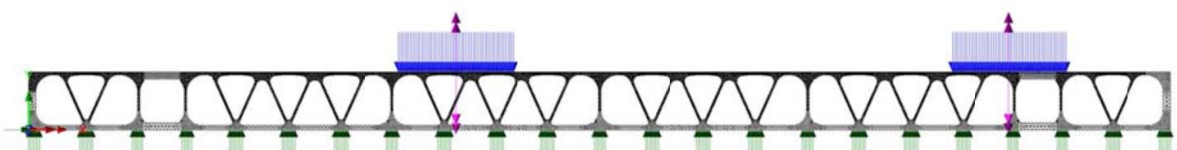


Figure 47 – System 3 Model

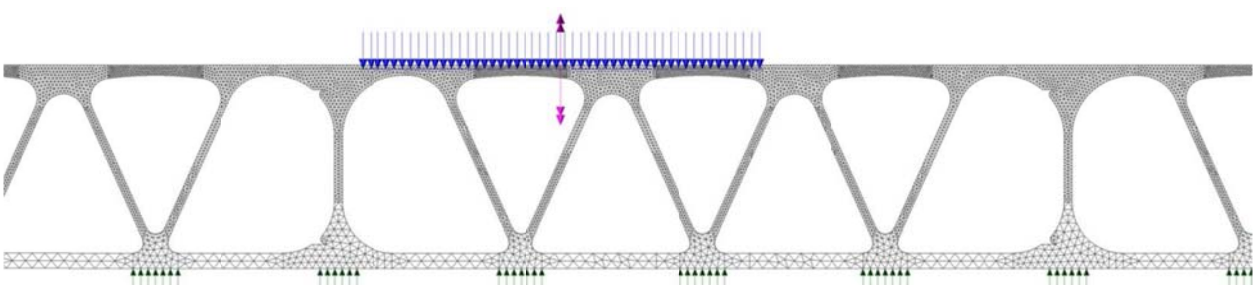


Figure 48 – System 3 Model Close-up

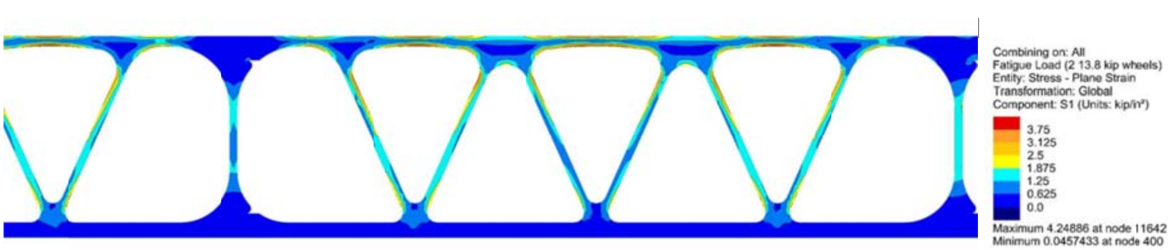


Figure 49 – System 3 Fatigue Analysis Results

2.3 DECK TO STRINGER CONNECTION

Deck to Stringer Connection: A slip-resistant connection for live loading is recommended for the deck to stringer attachment as the high number of repeated cycles of slip (i.e. in the tens-of-millions over a 25 to 75 year period) between the deck and stringers may cause fretting that could wear protective coatings on the stringers needed to prevent corrosion. Analysis demonstrates that it is practical to design a slip-resistant connection with a reasonable number of fasteners to resist slip from live load.

However, the combination of thermal forces due to the difference in coefficient of thermal expansion between aluminum and steel and forces due to live load are too large to design a practical slip-resistant connection. Slip can be permitted under thermal loading as the number of cycles of thermal loading that cause slip is low. Once slip due to thermal loading occurs, the built-up forces are relieved until the next significant thermal variation occurs. Significant changes in temperature large enough to cause slip tend to be more seasonal and not daily. Even if slip occurred daily, the number of cycles is low (i.e. in the tens-of-thousands over a 25 to 75 year period) and not anticipated to cause fretting.

In order to make the build-up of thermal forces more manageable for the stringer-to-floorbeam end connection design, sealed expansion joints are recommended between at the floorbeams and at midway between the floorbeams.

To resist slip, deck panels are fastened to the top flange of the stringers with a bolted connection using fully pre-tensioned, conventional 3/4" diameter ASTM A325 bolts of heavy hex (HH) or tension control (TC) type. Bolts are to be inserted in standard oversize (15/16" diameter) holes in both the deck plate and stringer flange. The faying surface between the aluminum deck and steel framing have been previously certified by the Research Council on Structural Connections (RCSC) as meeting the requirements for a ASTM A325 Class 'B' (0.5 coefficient of friction).

The shear flow between the deck and stringers is computed using manual calculations (e.g. spreadsheet calculations.) In the calculations, the deck is considered fully composite with the stringers but with discontinuity at the expansion joints. In addition, the deck bottom plate is bolted directly to the stringer top flange, while the top plate is only attached to the bottom plate by a series of inclined and vertical web members. The above features introduce shear lag effects, which introduce variations in the composite section properties along the length of the stringers. The deck is fully non-composite at the expansion joints and the effective width of the deck increases away from the joints until the full effective width is achieved. Because the top plate is not directly connected to the stringer and is only indirectly connected by the deck web members, additional shear lag effects must be considered for the top plate.

A finite-element analysis confirmed that the bottom plate effective width increases linearly from the expansion joints at 45 degree angle each side of the axis of the stringer (i.e. effective width = 2 x distance from the joint.) The top plate effective width increases at a slower rate equal to the square of the fraction of the bottom plate effective width to the full composite width. (See Figure 53)

The non-prismatic features introduce differences in how the shear flow between the deck and stringers is calculated compared to typical composite deck and stringer systems, where the shear flow is computed simply as VQ/I. With the aluminum orthotropic deck, shear flow must be determined by computing the rate in change (i.e. slope) of the deck compressive force along the length of the stringers. The compressive force in the deck is computed using AASHTO LRFD live load moments at the Service II Load Combination (both Truck and Tandem Loading with Lane Component), and composite section properties as described above (i.e. based on effective width of the deck, transformed area, and strain compatibility.)

The increase in bending moment at the center of the stringer and the discontinuity in the deck at the mid-span expansion joint, results in a rapid rate of change in deck compression approaching the discontinuity with a corresponding increase in shear flow. (See Figures 50 thru 52)

The pitch of the bolts is a function of the voids in the deck panel, which are typically at 4½" on center, At the end extrusions, bolts must be omitted from the bay with the thicker top and bottom flange. Bolts can be provided each side of the stringer web and at each of the voids if necessary or can be omitted from voids as permitted by calculation. The required pitch can be reduced by increasing the stiffness (i.e. moment of inertia) of the stringer or reducing the stringer spacing.

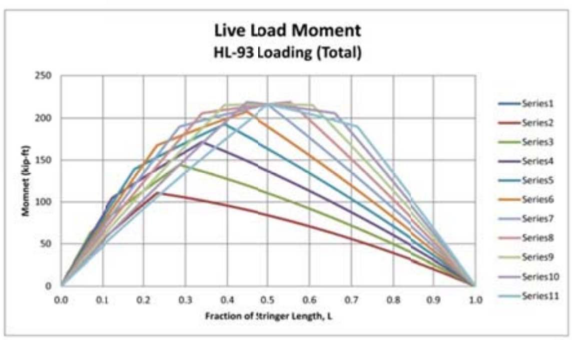


Figure 50 – Stringer Live Load Moments

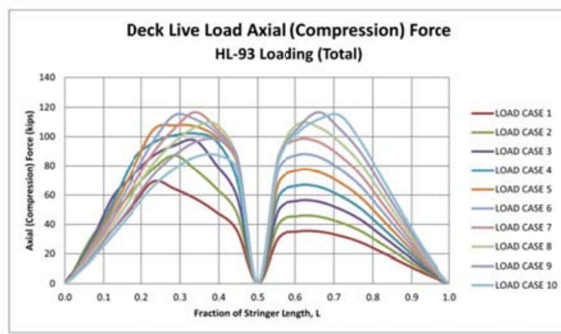


Figure 51 – Deck Compressive Forces

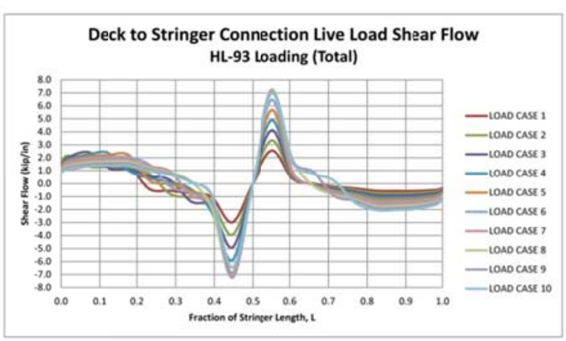


Figure 52 – Deck-to-Stringer Shear Flow

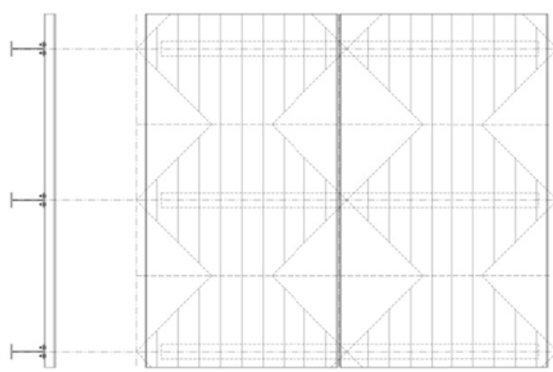


Figure 53 – Deck Effective Width

2.4 STRINGER DESIGN

Stringer Member: The stringers can be designed accounting for the composite behavior of the deck or can be conservatively designed as non-composite. The stringer is typically designed conservatively as a simple-span between floorbeams. Stringers should be analyzed for AASHTO LRFD Service II (Deflections), Strength I and Strength II Load Combinations. AASHTO LRFD Live Loads should include Truck and Tandem vehicles in combination with lane components as is customary.

Stringer End Connections: The stringer end connections should de designed for combined shear and resisting end moments. Although stringers are customarily considered simple-span for sizing the member, the end connections provide significant moment resistance due to the resistance of the stringer on the opposite side of the intermediate floorbeam. The stringer end connections should consider the effects of live load and thermal effects due to the difference in coefficients of thermal expansion between aluminum and steel including both thermal rise and fall. [NOTES: Attempts were made to introduce a more flexible end connection that would relieve end moments due to thermal effects. However, the more flexible end connection lacked sufficient capacity to also support live load.]

The proposed stringer connections have been designed to accommodate accelerated bridge construction. The intent is for the deck to be attached to the stringers in the shop and ship the deck-stringer unit as an assembly. In order to erect the assembled unit, the stringers must clear the existing floorbeam top flange. As such, the stringers are cut short, a tee section bolted to the web of the floorbeam with the stem of the tee aligned with the web of the stringer, and connection plates used each side of the stringer web similar to a web splice. (See Figures 7 and 8)

Because the aluminum orthotropic deck has the ability to span greater distances than steel open grid deck, the stringers can be respaced at a greater spacing. The respacing permits the stringers to be offset from the existing stringers so that the holes of the existing connections can be avoided. It also permits the new tees to be installed prior to removing the existing steel open grid deck and stringers from service.

3.0 PROPOSED TEST PROGRAM

<u>Phase 1 – Component Testing</u> : The deck system is to be tested at Florida Department of Transportation, Structures Research Center, Tallahassee, Florida. (See Test Program Sketches in the Appendix.) The purpose of Phase 1 of the laboratory load test program is generally to:

1. Verify performance and response of individual deck panels of similar size and boundary conditions to those proposed for actual in-service conditions, and under loading configurations and magnitudes consistent with AASHTO LRFD.
    a. Measured stresses and deflections will be compared with AASHTO LRFD limits. The comparison is used to confirm the structural adequacy of the proposed deck design.
    b. Measured stresses and deflections will be compared with values computed from finite element analysis. The comparison is used to confirm the validity of the finite element analysis so that the finite element analysis can be used to accurately evaluate additional loading scenarios not evaluated in the testing as required.
    c. Measured stresses and deflections will be compared with those computed using manual calculations based on use of AASHTO LRFD equivalent strip widths and shear lag effects based on effective flange width concepts.
2. Verify performance of the proposed wearing surface when the panels are subject to design level loading. The wearing surface is to be included on the test panels during the testing.

3. Establish temperature gradient design values for the aluminum deck panel and steel stringer system. Measure thermal variations within the deck panel and stringer during full range of daily temperature variations to establish values to be used in design.
4. Verify performance of proposed conventional ASTM A325 high-strength fasteners (heavy hex type and tension control types) and Lindapter Hollo-Bolt High-Clamping Force blind-type fasteners and for attachment of the deck to the supporting steel framing and performance of the deck panel at the connection. Measure static tension strength, sustained tension proof load, static shear strength, and slip-critical shear resistance for each of the fastener types using supplemental test pieces of the deck. Evaluate constructability of each of the fastener types.
5. The test set-up includes two test panels measuring 7'-7½" x 14'-0" (Gen I Panel) and 8'-3" x 14'-0" (Gen II Panel) mounted to three W16x36 Stringers spaced at 6'-0" on center with 1'-4½" and 7½" cantilevers, each side respectively. The deck is attached to the stringers with a series of ¾" ASTM A325 Tension Control (TC) Bolts. The stringers are painted with an inorganic zinc primer and the bottom of the aluminum panels are abrasion blasted to achieve a Class 'B' Slip Surface. The test set-up has specifically been configured for use in Phase 2 – In Service Structural System Testing. (See Figures 18 thru 23) The shop fabrication will demonstrate constructability.

<u>Phase 2 – In Service Structural System Testing</u>: Following successful Phase 1 - Component Testing, the Department may consider an additional expanded test program that tests and evaluates the response and performance of the full structural floor system under actual in service loads on an existing bascule bridge. (See Test Program Sketches in the Appendix.) Details of Phase 2 – In Service Structural System Testing are generally anticipated to include the following:

- The recommended test configuration will consist of a single-lane of one intermediate floorbeam bay of a typical bascule leaf. North Causeway Bridge (940045) in Ft. Pierce, Florida, has been identified for this purpose. The test can make use of the Phase 1 – Components Testing test set-up described above. The stringers will frame into three W30x108 (or similar) floorbeams spaced at 16'-9" on center using the proposed stringer to floorbeam end connection details described above. The existing steel open grid and stringers in this bay will be removed as a single unit and stored for future use if needed. Trim pieces will be added to the existing steel open grid deck and the center stringer modified to facilitate the new aluminum deck unit.
- The deck system will be initially tested by the Department's Load Rating Trucks. Upon successful performance the lane will be reopened to vehicular traffic of in service testing.
- Measured stresses and deflections from the Phase 2 – In Service Structural System Testing will be compared with those from the Phase 1 – Component Testing.
- The overall performance of the structural system including deck panels, wearing surface, deck to stringer connections, and stringer to floorbeam connections will be evaluated.
- The constructability of the structural system will be further demonstrated and evaluated.

3.1 DESCRIPTION OF TEST PANELS AND SUPPLEMENTAL TEST PIECES

AlumaBridge, LLC has agreed to provide two (2) panels and supplemental test pieces for the testing. The first panel consists of Gen I deck panel that measures 7'-7½" wide x 14'-0" long fabricated by splicing together six (6) 1'-1½" wide Primary Extrusions and two (2) 5¼" wide End Extrusions using two-sided, self-reacting friction stir welding. The second panel consists of Gen II deck panel that measures 8'-3" x 14'-0" long fabricated by splicing together three (3) 1'-6" wide Male-Female Primary Extrusions, one (1) 1'-6" wide Male-Male Primary Extrusions, and two (2) 1'-1½" wide End Extrusions. The Gen I aluminum extrusion profiles were extruded at SAPA in Connersville, Indiana and the Gen II aluminum extrusion profiles were extruded by Taber Extrusions, LLC, Russellville, Arkansas. Panels were fabricated by friction stir welding at HF Webster in Rapid City, South Dakota. The wearing surface was applied to the panels at HRI, Inc. in Williamsport, Pennsylvania. (See Test Program Sketches in the Appendix.)

Supplemental test pieces for additional use and testing consist of a series of 1'-0" long partial deck panels cut from the individual Gen I Primary Extrusions (12 Req'd).

3.2 PANEL COMPONENT STATIC LOAD TESTING

A series of load tests are proposed using the test set-up described above. Loading is to be applied to the Gen II Panel only. (See Test Set-up Drawings in Appendix.) The panel is to receive a series of static loads in different magnitudes and configurations to simulate the governing AASHTO LRFD design loadings for the Service I, Service II, Strength I and Strength II Load Combinations. Biaxial strain gauges and deflection gauges placed at specified locations will be used to record the panel stresses and deflections. (See Test Program Sketches in the Appendix.)

2.5 CONNECTION TESTS

<u>Fasteners</u>: The primary purpose of the fasteners is to connect the deck to the supporting steel framing. The design of the deck and floor system does not rely on composite behavior between the deck and supporting stringers. However, it is recognized that some composite behavior is inherent in this connection. Prevention of slip (relative movement between the deck and stringers) at this connection under traffic loading is generally desirable for serviceability reasons as repeated slip between the deck and stringers under traffic loading may cause undesirable fretting that can result in premature deterioration of protective coatings at the faying surfaces. As such, it is preferable that the fasteners resist slip under live loads at the Service II Limit State by way of adequate clamping action produced by sustained preload from properly tensioned high-strength fasteners. Due to the significant difference in coefficients of thermal expansion between aluminum and steel, large temperature variations will yield relative movement between the aluminum deck and steel framing that can also result in slip at the faying surfaces of the connection. Because large variations in temperature typically occur slowly over long periods of time, the number of cycles of slip caused by large temperature variations is anticipated to be low.

Conventional High-Strength Fasteners: AASHTO LRFD recognizes and permits use of ASTM A325 high-strength bolts in slip-critical connections for aluminum structures. ASTM A490 high-strength bolts are not permitted for aluminum structures. Hardened washers are required between the bolt head or nut and the aluminum surface to prevent galling and relaxation. Sustained proof-load values for ASTM A325 high-strength bolts in aluminum structures using ASTM B221 Alloy 6063-T6 are the same as those for steel structures, with no measurable relaxation. Pre-tensioning of high-strength fasteners prevents fasteners from loosening under vibration.

Laboratory testing is to be performed to verify the design values in AASHTO LRFD for ASTM A325 high-strength bolts used in the aluminum deck to steel stringer connection. As the testing is to be performed using pieces of the aluminum extrusion, the testing will also be used to evaluate the constructability of the bolted connections (i.e. bolt installation and tensioning) and performance of the deck panel at the connections.

Blind-Type Fasteners: Currently, *AASHTO LRFD Bridge Design Specifications*, *AASHTO LRFD Bridge Construction Specifications*, and the FDOT *Structures Design Guidelines* and *Standard Specifications for Road and Bridge Construction* do not address use of blind-type fasteners. As such, laboratory testing is recommended to verify that the fasteners will adequately perform as intended in this application and to establish parameters that can be used in development of design and construction specifications.

Blind-type fasteners should be load tested and evaluated for the following:

1. Static Shear Strength (Bearing)
2. Static Tension Strength
3. Static Tension Proof Load
4. Static Slip-Critical Shear Resistance.

Based on discussions with the District 4 Bridge Maintenance Office, only one type of blind-fastener will be considered: 3/4" Lindapter High-Clamping Force Hollo-Bolts (LHB-HCF) (Product Code LHBM20#1 Hexagonal Stainless Steel). The LHB-HCF fastener can be field installed to anchor the hollow aluminum deck panels if required to retrofit a connection. Because sustained pre-tensioning proof loads for this type of fastener is significantly lower than similarly sized conventional ASTM A325 bolts, the bolt is not recommended as the primary fastener type in a slip-resistant connection. The bolt is only recommended in maintenance applications to replace a limited number of conventional ASTM A325 bolts, where required. The fastener is installed in 1-3/8” diameter standard size holes (1/16” in diameter larger than the split-sleeve).

LHB-HCF were previously tested and evaluated for use in structural steel connections by ICC Evaluation Service, LLC (ICC-ES), subsidiary of the International Code Council. The testing and evaluation of this fastener and connection was performed with the purpose of obtaining recognition in the *International Building Code (IBC)*. The procedures and acceptance criteria for the testing and evaluation are described in *Report AC437 – Acceptance Criteria for Expansion Bolts in Structural Steel Connections (Blind-Bolts)*. No testing and evaluation of the LHB-HCF has been performed for aluminum to steel connections. It is recommended that testing and evaluation be performed for LHB-HCF in aluminum to steel connection using similar protocol described in Report AC437 and as described below.

The protocol described in Report AC437 evaluates the fasteners to the provisions of the following AISC Specifications:

- AISC 360 – Specification for Structural Steel Buildings
- AISC 348 – Specification for Structural Joints using ASTM A325 or A490 Bolts
- AISC 341 – Seismic Provisions for Structural Steel Buildings.

Given the intended use, it is not necessary to evaluate the performance of the fasteners for the provisions of AISC 341 at this time.

The testing and evaluation criteria generally assess the strength and deformation capacity, service conditions, design procedures and quality control.

Lindapter reports that LHB-HCF type blind-fasteners can be used as an alternative to bolts conforming to ASTM A325 Specifications. LHB-HCF fasteners produce a pre-tensioned connection, although the reported sustained pre-load values are significantly lower than those for standard high-strength steel connections using ASTM A325 bolts. Published information for steel-on-steel connections lists a an initial pre-load for 3/4" diameter LHB-HCF fasteners of approximately 14.0 kips and initial loss in pre-load of approximately 4.0 kips for a net sustained preload of 14.0 kips (approximately half of typical preload for ASTM A325 of 28.0 kips.)

Fastener installation and tensioning is performed in accordance with Manufacturer's published installation instructions using a torque wrench to produce controlled tension forces on the bolt to the requirements of AISC Specification 348.

Elements to be fastened to each other shall be aligned with uniform bearing and no significant gap between faying surfaces at the time of bolt tensioning. Clamps or preloading shall be used as required to eliminate gaps.

LHB-HCF bolts are installed in standard holes as defined in AISC Specification 360, with diameters no greater than the blind-bolt shell diameter plus 1/16". Burrs in the holes shall be removed before inserting the LHB-HCF bolts. Holes are to be drilled from solid with the components in assembly and in the required alignment. Alternatively, holes can be pre-drilled in the steel support stringer flange and the holes used as a template to drill the holes in the aluminum orthotropic deck bottom plate. Holes shall be temporarily aligned with a mandrel (i.e. tapered pin) as required during bolt installation.

In order to establish design values, the protocol described in Report AC437 specifies a minimum of three tests for each type of test (shear strength, slip-critical shear resistance, and tension strength), and each size, length and material (including finishes) for the proposed fasteners. In general, the grip length for the structural connections is anticipated to be relatively consistent (i.e. approximately 1/2" +/- 1/8" thick for typical stringer flange plus approximately 3/8" thick for aluminum orthotropic deck bottom plate.) As such, it will only be necessary to test a single length fastener for each type of blind-fastener. Based on preliminary analysis of the connections and due to dimensional restrictions associated with the aluminum orthotropic deck, only 3/4" diameter fastener sizes will be considered. Due to the need for a pre-tensioned connection only high-strength material (ASTM A325/SAE Grade 5 material or similar) will be considered. Due to the anticipated salt-water environment, corrosion resistant material or corrosion resistant finishes will be used. Type 316 stainless steel is available for the LHB-HCF fastener components. Given the above considerations, there will be only one size, length and material for each type of fastener that requires testing.

The tested capacity from each of the test protocol shall be the average peak strength of all tested values. The tested capacity shall be adjusted downward from the actual measured ultimate strength of the test specimens if the results exceed that of the minimum specified strength of any of the individual components including that of the aluminum orthotropic deck components.

For the LHB-HCF fasteners, the adjusted fastener capacities for shear and tension are as follows:

Adjusted Shear Capacity = Tested Shear Capacity x α_v, where:

$$\alpha_v = \frac{\left[D_c^2 F_{uc} + \left(D_{S2}^2 - D_{S1}^2\right) F_{us}\right]_{Specified}}{\left[D_c^2 + \left(D_{S2}^2 - D_{S1}^2\right) F_{us}\right]_{Actual}} \leq 1.0$$

Adjusted Tension Capacity = Tested Tension Capacity x α_t, where:

$$\alpha_t = \frac{\left[D_c^2 F_{uc}\right]_{Specified}}{\left[D_c^2 F_{uc}\right]_{Actual}} \leq 1.0$$

and where:

D_c denotes Core Diameter

F_{uc} denotes Core Ultimate Strength

D_{s1} denotes Shell Inner Diameter

D_{s2} denotes Shell Outer Diameter

F_{us} denotes Shell Ultimate Strength

The nominal capacity, R_n, of the fastener shall be the lowest adjusted tested capacity from the tests performed.

All components of the testing apparatus shall have capacities that exceed the ultimate capacity of the fastener for the test type in question.

A resistance factor, φ, shall be determined and applied to the nominal capacity for determining the AASHTO LRFD load and resistance factor strength. The resistance factor shall be computed per Chapter F of AISI S100 as follows:

$$\varphi = 1.672e^{-4.0\sqrt{0.053 + C_p V_p^2}}$$, where

V_p = Coefficient of variation of test results ≥ 0.065

$C_p = 5.7$ for $\mathrm{n} = 3$

$\mathrm{n} =$ Number of tests

Static Tension Test: Each fastener type shall be tested to determine tension strength and to verify proof-load in an assembly using the following approach:

For each test, a single blind-fastener unit shall be installed in a Tension Calibrator (a.k.a. Skidmore-Wilhelm Calibrator) as an assembly with an aluminum plate equal to the thickness of the aluminum orthotropic deck bottom plate (3/8”), a steel plate equal to the thickness of the top flange of the steel support stringer (1/2”), and with fastener installed per the Manufacturer’s recommendations.

Initial assemblies shall be tested to failure as described below to determine the ultimate strength of the blind-fastener assembly in tension.

1. The blind-fastener shall be initially fastened in a snug-tight condition in accordance with Section 8.1 of AISC 348.
2. A continuous monotonic load shall be increasingly applied at a rate ranging from 25 percent to 100 percent of the blind-fastener anticipated ultimate tension strength until one of the following occurs:
    a. There is a sudden failure or zero measured load resistance.
    b. There is a 20 percent or greater reduction in measured load resistance.
3. Load application for each test shall be performed over a period of time no less than one minute.

For purpose of estimating a maximum anticipated test load, the ultimate capacity of the bolt in tension shall be calculated in accordance with Section J3.6 of AISC 360. For the LHB-HCF fasteners, only the contribution from the bolt core shall be considered with no contribution from the sleeve.

After failure, examine the assembly and report the governing mode(s) of failure of each assembly.

After the ultimate strength of the blind-fastener assembly in tension is determined, verify tension proof load of the assembly, using a methodology similar to that used to determine the ultimate strength, with the assembly loaded to a maximum tension equal to 70 percent of the measured ultimate strength in tension.

1. The blind-fastener shall be initially fastened in a snug-tight condition in accordance with Section 8.1 of AISC 348.
2. A continuous monotonic load shall be increasingly applied at a rate ranging from 25 percent to 70 percent of the blind-fastener anticipated ultimate tension strength.
3. Load application for each test shall be performed over a period of time no less than one minute.
4. The tension proof load shall be sustained for a minimum of five days to determine the amount of relaxation in the assembly, if any.

Static Shear Test: Each fastener type shall be tested for shear in an assembly using the following approach:

The fasteners shall be installed in a four bolt, single shear lap type joint assembly using the aluminum orthotropic deck test pieces and the top flange of the steel support stringer.

A displacement measuring device shall be positioned to measure the shear displacement at the interface between the aluminum orthotropic deck test piece and the steel support stringer in the direction of loading.

For purpose of estimating a maximum anticipated test load for the LHB-HCF fasteners, the fastener may be assumed to be a single solid shaft with a nominal diameter equal to the outer diameter of the shell and ultimate stress of either the shell or outer core.

1. The connection shall be fastened in a snug-tight condition in accordance with Section 8.1 of AISC 348.
2. An initial load equal to five present of the bolt anticipated ultimate shear capacity shall be applied in order to bring all members into full bearing.
3. A continuous monotonic load shall be increasingly applied at a rate ranging from 25 percent to 100 percent of the bolt anticipated ultimate shear strength until one of the following occurs:
    a. There is a sudden failure or zero measured load resistance.
    b. There is a 20 percent or greater reduction in measured load resistance. This reduction shall not be attributed to slip or sudden loss in friction strength as verified by the measured displacement. Loading shall continue if reduction is attributed to slip.
4. Load application for each test shall be performed over a period of time no less than one minute.

After failure, examine the assembly and report the governing mode(s) of failure of each assembly.

Static Slip-Critical Shear Test: Each fastener type shall be tested for slip-critical shear in an assembly using the following approach:

The fasteners shall be installed in a four bolt, single shear lap type joint assembly using the aluminum orthotropic deck test pieces and the top flange of the steel support stringer. The faying surfaces shall be prepared for a Class B surface (0.50 mean slip coefficient.) [NOTE: The use of Class B is consistent with AISC 360 Section J3.8, ADM Section J3.8, and previous laboratory testing of abrasion blast cleaned aluminum in contact with hot-dip galvanized or zinc-rich painted steel surfaces. Tests of mill finish aluminum surfaces that have been degreased and dried exhibited relatively low coefficients of friction, and thus mill finished aluminum surfaces should not be used.]

A displacement measuring device shall be positioned to measure the shear displacement at the interface between the aluminum orthotropic deck test piece and the steel support stringer in the direction of loading.

The fastener shall be pre-tensioned to the established tension proof load values from the previously performed Static Tension Tests and in accordance with the Manufacturer's recommended installation methods. If the blind-fastener is subject to relaxation, as determined in the Static Tension Tests, the minimum time for full relaxation to occur shall lapse between the time the fasteners are pre-tensioned and the testing to allow relaxation to occur.

The anticipated slip friction capacity may be calculated in accordance with Section J3.8 of AISC 360 with the following parameters:

μ = 0.50 assuming Class B surface

D_u = 1.13 bolt pretension factor

h_{sc} = 1.0 for standard size hole

N_s = 1.0 for a single slip plane

T_b = pretension value.

1. A continuous monotonic load shall be increasingly applied at a rate equal to the lesser of 25 kips per minute or 0.003 inch of slip displacement per minute until any of the following occurs:
    a. There is 0.05 inch of displacement.
    b. There is a 20 percent or greater reduction in measured load resistance.
    c. There is a sudden failure or zero measured load resistance.
2. Load application for each test shall be performed over a period of time no less than one minute.