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Aluminum Orthotropic Deck: A Viable Alternative to Steel Open Grid Deck (at Last)

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Introduction

General

Owners and users of movable bridges with steel open grid deck have long dealt with the numerous maintenance and performance issues with this lightweight deck system. The decision to originally use steel open grid deck was primarily to reduce weight and thus reduce load on the structure and operating equipment with corresponding cost savings. At the time the majority of these bridges were constructed, steel open grid deck was considered the only available and practical lightweight deck system. Steel open grid deck has been problematic for a number of reasons including:

- Openings in the deck permit dirt and debris to collect on the steel framing members below. The dirt and debris retains moisture that contains chlorides from the saltwater environment or deicing salts, which is conducive to corrosion development. The network of steel grid bars makes cleaning of the steel framing members difficult.
- Although the deck typically includes a serrated top surface to improve skid resistance, the top surface polishes over time from contact with wheels, which eventually reduces the skid resistance, especially when wet, and reduces safety.
- The welded fabrication of the steel open grid deck includes numerous fatigue sensitive details (Category E per AASHTO LRFD) that are prone to fatigue development and as such the deck design is typically governed by fatigue provisions. Bridges with heavier truck traffic have commonly experienced premature fatigue cracking and localized failure of the secondary and tertiary bars that result in holes in the deck.
- Fabrication tolerances of both the steel open grid deck and bascule leaf steel framing have resulted in difficulties in achieving uniform bearing of the deck main bars on the supports. Excessive root openings and poor field welding practices have resulted in widespread cracking of the deck attachment welds.
- Tires in contact with the network of steel open grid bars and corresponding openings between the bars create resonant vibrations that generate noise that is considered a nuisance to residences and businesses nearby these bridges.
- The relatively large openings in the deck and the slippery surfaces make crossing the movable span on a bicycle a challenge. The bicycling community considers steel open grid deck a safety concern.

Stakeholders have sought alternative solutions to steel open grid deck that offer a similar weight, but include a solid surface and other features that address these concerns. Thanks to a Florida Department of Transportation research and development project, together with advancements in friction stir welding, prior research from the 1990's performed by Reynolds Aluminum, FHWA, Virginia Department of Transportation and Virginia Tech, and a new aluminum orthotropic deck product offered by AlumaBridge, LLC, that day may finally be here.

This paper discusses the research program that led to development of a new lightweight 5-inch aluminum orthotropic deck system with a solid surface that can replace steel open grid deck on a weight neutral basis. Also included in this discussion are:

- Deck features
- Deck panel fabrication using friction stir welding
- Deck to steel framing connections,
- Design methodology
- Corrosion resistance
- Thermal effects
- Wearing/friction course
- Deck cross slope and drainage
- Traffic railings
- Strategies and details for accelerated bridge construction (ABC).

Alternatives Evaluation

Screening

The Florida Department of Transportation research project started with an in-depth screening of available lightweight bridge deck systems. There are a number of factors that should be considered in the selection of a lightweight solid deck system including:

- Deck Weight/Load Capacity Limitations:
    - Main longitudinal load carrying member strength
    - Trunnion shaft strength, fatigue resistance, and deformation
    - Trunnion bearing resistance
    - Rolling-lift bascule segmental girder track girder, track and tread strength
    - Vertical-lift wire ropes and sheave strength
    - Swing pivot bearing resistance
    - Counterweight adjustment and connection strength.
- Adaptability to Varying Dimensions and Arrangements:
    - Deck system depth (with or without purlins)
    - Span length and roadway width
    - Steel framing configurations
    - Deck cross slopes
    - Joint configurations
    - Traffic railings and curbs.
- Costs:
    - Deck fabrication and installation,
    - Modifications to movable span steel framing, piers, approach spans, counterweights, and support machinery
    - Design
    - Construction inspection
    - Future maintenance and inspection.
- Functionality and Safety:
    - Maintain or improve load capacity
    - Improve skid/slip resistance for vehicular and bicycle traffic
    - Reduce traffic generated noise.

- Maintenance:
    - Ease of repair and/or replacement of all or portions of the deck
    - Need for periodic maintenance such as reapplication of coatings and/or replacement of wearing surfaces.
- Service-life and Durability:
    - Deck and/or deck component service-life
    - Resistance to corrosion, fatigue, wear, impact, fire, ultraviolet light, and chemicals
    - Accommodation of thermal movements.
- Constructability:
    - Disruption to traffic
    - Ability to accommodate fabrication and installation tolerances
    - Sensitivity to environmental conditions during construction
    - Shipping, storage and handling
    - Specialized inspection requirements during fabrication and installation.
- Design Methodology:
    - Availability of simple closed form equations in lieu of finite element analysis
    - Procedures included within AASHTO LRFD Bridge Design Specifications
    - Technology familiar to the bridge design community
- Other Risks including:
    - Familiarity of product and technology including years' experience, previous bridge installations, quantity and quality of applicable research, endorsement by bridge design community and AASHTO
    - Availability of design tools and construction techniques
    - Financial and technical support from supplier(s)
    - Product availability (e.g. opportunities for competitive bidding, sole source and/or patents).

In the past 20 years, new materials and technologies have developed that introduce new lightweight solid deck systems including:

- Aluminum orthotropic deck
- Sandwich plate system (SPS) deck
- Fiber reinforced polymer (FRP) deck
- Ultra-high performance concrete (UHPC) waffle slab deck
- Steel orthotropic deck
- Reinforced concrete slab with lightweight concrete
- Partially filled steel grid deck with lightweight concrete
- Exodermic deck with lightweight concrete

The deck systems were initially screened for unit weight. Typical hot dip galvanized 5-inch 4-way steel open grid deck of welded construction, found on most movable bridges today, has a basic unit weight of 21 psf (standard-duty). For the initial screening, a maximum basic unit weight threshold of 25 psf was used, recognizing that there are opportunities to offset a slightly heavier deck by replacing other elements of the bridge with lighter weight components, and taking into account increased structural efficiency. Steel orthotropic, reinforced concrete slab, partially filled steel grid, and Exodermic decks have significantly heavier unit weights (greater than 50 psf) and thus were eliminated from further consideration. SPS deck and UHPC waffle slab deck were later found to have unit weights that were significantly more than 25 psf during the evaluation phase, when considering all of the details.

A Value Engineering (VE) approach was used to provide a reasonable, quantifiable evaluation and comparison of the different alternatives. The existing steel open grid deck was used as the baseline for the evaluation and comparison. The alternative lightweight solid deck systems (aluminum orthotropic, SPS, FRP, and UHPC waffle slab decks) were scored and compared to steel open grid deck and to each other for 40 different criteria to determine which deck offered the most value. In the VE approach, each of the evaluation criteria was assigned an importance factor.

Selection

The aluminum orthotropic deck scored far above steel open grid deck and the other deck systems and thus was selected by the Department for further research and development. This lightweight solid deck system is recommended for the following reasons:

- The solution will address all of the functionality and safety concerns of the steel open grid deck.
- It provides a nearly weight neutral solution that can be implemented on most typical movable bridges with minimal changes in overall weight.
- Relatively adaptable to different bridge configurations and dimensions with minimal bridge modifications and accommodates accelerated bridge construction. The deck is fabricated in panels similar to steel open grid deck and can be replaced with short duration closures of the bridge or phased construction while traffic is maintained.
- The aluminum orthotropic deck is anticipated to be durable and provide a long service life with excellent resistance to corrosion and other environmental factors, and will better protect the steel framing. The deck is conservatively designed for infinite fatigue resistance and the friction-stir welding technology improves fatigue resistance. Aluminum does not require protective coatings.
- Temporary short-term repair of the deck top surface is relatively simple and can be performed rapidly with the addition of field welded aluminum plates.
- The proposed deck is generally of robust design (high strength and stiffness due to efficient configuration) that conservatively meets all Strength, Service (deflection) and Fatigue Limit States in the *AASHTO LRFD Bridge Design Specifications*. Updated design specifications for the aluminum orthotropic deck, which utilize simple, conservative closed-form equations, have been ratified by AASHTO and incorporated into the *AASHTO LRFD Bridge Design Specifications*.
- A significant amount of research, development, testing, analysis and evaluation of the aluminum orthotropic deck has been performed including:
    - An extensive Department of Energy – Oak Ridge National Laboratory test program on an 8-inch deep deck performed by Reynolds Metals Company, FHWA, Virginia DOT, and Virginia Tech University in the 1990's with participation by notable engineers including:
        - John Ahlskog, former Assistant Chief Bridge Engineer, FHWA
        - John Kulicki, Modjeski & Masters
        - Randy Kissell, primary author of *AASHTO LRFD* Section 7 (Aluminum)
        - Roman Wolchuk, renowned orthotropic bridge engineer
    - A more recent test program by Florida DOT for the 5-inch deep deck.
- Aluminum orthotropic bridge deck has a long history of successful performance including:
    - Smithfield Street Bridge, Pittsburgh, PA with original installation in 1936, replacement in 1967 with more corrosion resistant alloy, and removal in 1995). This deck served as the basis for current designs.
    - Route 58 over Little Buffalo Creek, Mecklenburg, VA, 1997, which is on a heavily traveled truck route that receives snow plowing.
    - Historic Corbin Suspension Bridge, Huntingdon, PA, 1996.
    - Route 974 over Howard Creek, Clark, County, KY, 2005.
    - Alan Road Bridge, Sandisfield, MA, 2012, which was the first friction stir welded deck.
    - St. Ambroise Bridge, Quebec, Canada, 2015.
    - More than 70 installations in Europe.

- The aluminum orthotropic deck has been used in a number of other similar structural applications including train car, cargo aircraft decks, and aircraft carrier landing platforms.
- The 5-inch deep aluminum orthotropic deck is a derivative of the 8-inch deep aluminum orthotropic deck, but is more conservative with similar material thicknesses (top, bottom and web) but with shorter distances between panel points.
- Aluminum is a relatively well known material with consistent and predictable material properties and quality control.
- Recent advancements in friction-stir welding have eliminated previous concerns with gas metal arc (MIG) welding.

As with all lightweight deck systems, there are also disadvantages and challenges that must be addressed:

- The deck system has a higher initial construction cost than steel open grid deck ($120 per square foot for panels delivered with wearing surface). However, it is anticipated that the deck system will yield lower maintenance costs and reduced user impacts (i.e. reduced user delay costs and accident cost) that will offset the higher initial cost.
- The hollow profiles complicate attachment details and require either blind type fasteners or fasteners that can be installed inside the hollow profiles and tightened from the outside. The connections have been thoroughly researched and there are simple, cost-effective bolted details that have been developed.
- Concerns with galvanic corrosion due to dissimilar metals has been researched, and proven corrosion mitigation strategies, including proper detailing, material selection, and coatings, have been identified.
- The deck requires a thin epoxy polymer wearing/friction course that must be maintained. The significant stiffness of the deck improves the durability and service life of the wearing/friction course by permitting use of an epoxy based polymer that has excellent wear resistance. A thorough test program has led to the development of detailed surface preparation and application specifications. Details have been developed to retain the wearing surface at the panel edges and ends along the joints.
- The coefficient of thermal expansion for aluminum is different than that of steel (12.8×10^{-6}/deg F for aluminum vs. 6.5×10^{-6}/deg F for steel). Strategies to address these effects have been developed, including connection details that accommodate the relative thermal expansion.
- There are limitations on deck panel sizes that require joints between panels. Simple, durable, cost-effective sealed joint details have been developed.
- With a solid surface, deck drainage must be addressed. Simple deck drain details have been developed that channels stormwater away from the steel framing. Strategies to accommodate deck cross slope including parabolic crown have been developed.
- Traffic railings may have to be supported on the aluminum deck. Although there has been no crash testing of traffic railings on the aluminum deck, calculations demonstrate that the deck has sufficient capacity to resist vehicular impact loads where steel post and beam traffic railings are through bolted to the deck.
- The aluminum orthotropic deck is a proprietary product, which can complicate procurement, especially on federally funded projects. However, all of the evaluated lightweight deck systems with solid surfaces were proprietary products.
- Although the proposed extrusions for the 5-inch deep aluminum orthotropic deck system were not available at the onset of this research, the extrusions have been engineered, are now available through AlumaBridge, LLC, and have undergone extrusion and friction-stir welding trials, and laboratory testing.

Deck System Description

Extrusions

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 extrusions have 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 with 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. End extrusions finish the ends of the panels and include a lip at the deck top to retain the wearing surface at the panel edge, and a lip to retain a 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. The two primary extrusion widths and variable width end extrusions allow for panels of any width. Extrusion trials confirm that the proposed profiles can be extruded to lengths up to 40 feet.

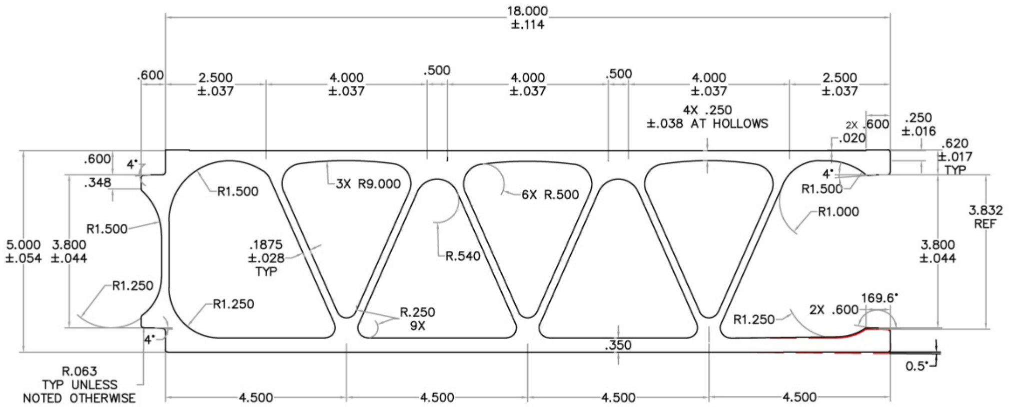

FIGURE 1: 5-inch Gen II Male-Female Extrusion

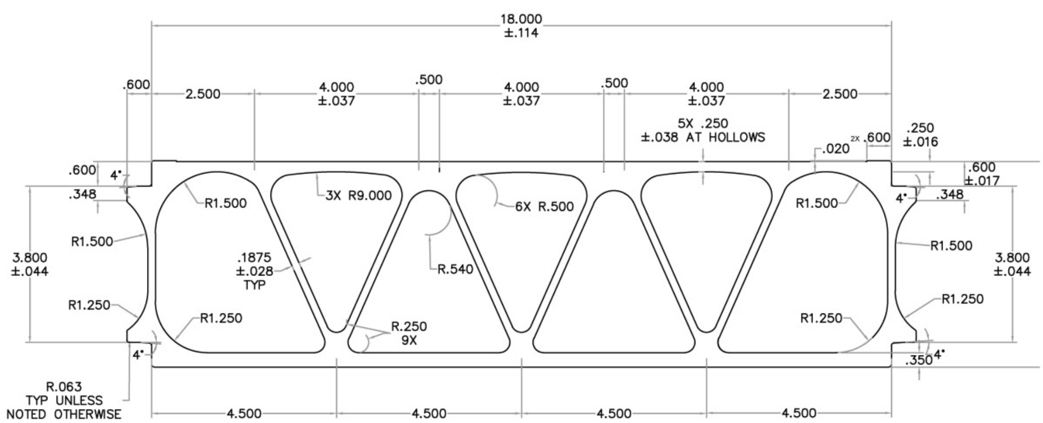

FIGURE 2: 5-inch Gen II Male-Male Extrusion

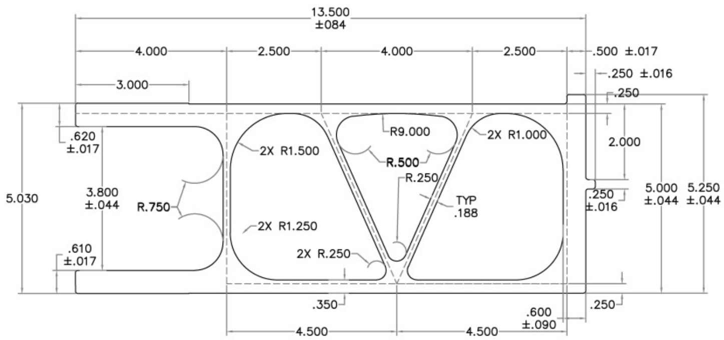

FIGURE 3: 5-inch Gen II End Extrusion

Deck Panels

The individual extrusions are spliced together using complete joint penetration single-sided FSW to create deck panels. FSW limits the width of panels to 14 feet.

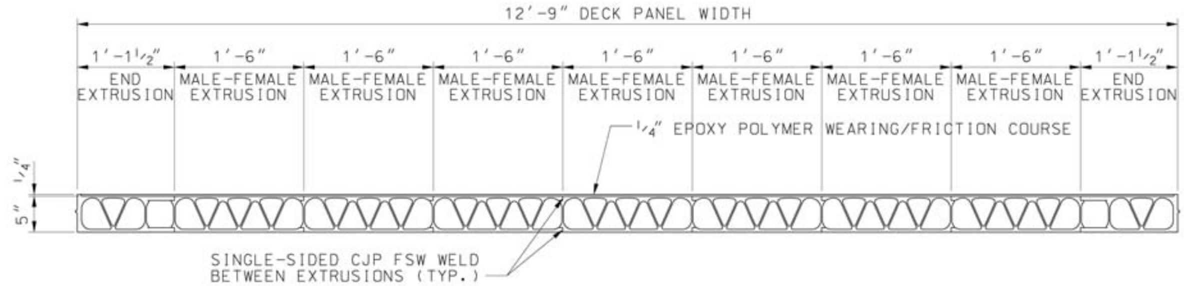

FIGURE 4: Typical Deck Panel

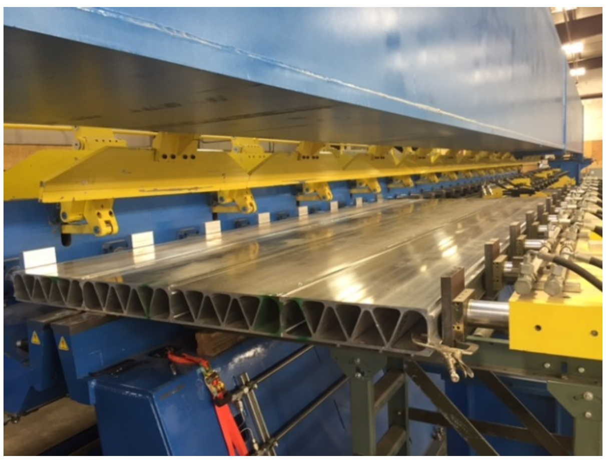
FIGURE 5: Friction Stir Welding Machine


FIGURE 6: Top Plate Friction Stir Welding

Unit Weight

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. With a 0.25-inch thick two-layer thin epoxy polymer wearing/friction course, the unit weight is approximately 20.9 psf. This unit weight is approximately equal to the weight of a hot dip galvanized standard duty 5-inch 4-way diagonal steel open grid deck. If a 0.375-inch thick three-layer wearing/friction course is used to increase the service life of the wearing/friction course, the unit weight increases an additional 2.0 psf.

Section Properties

The deck section properties are different in the primary direction (parallel to the extrusions) and secondary direction (perpendicular to the extrusions) as follows:

- *Stiffness*: The moment of inertia in the secondary direction is approximately 90% of the stiffness in the primary direction, making the deck panels nearly isotropic. The 5-inch aluminum orthotropic deck has significantly greater stiffness than the 5-inch steel open grid deck. Although the modulus of elasticity of aluminum is roughly one-third that of steel, the moment of inertia of the aluminum orthotropic deck is nearly six times that of standard duty 5-inch 4-way steel open grid deck, due to the significantly greater efficiency of the deck section.
- *Distribution Width*: The equivalent strip width of aluminum orthotropic deck is significantly greater than that of steel open grid deck. For decks supported on longitudinal stringers (transverse deck span) spaced at 63 inches on center, similar to typical bascule bridges in Florida with 5-inch steel open grid deck, the equivalent strip width for the aluminum orthotropic deck is 1.5 times that of the steel open grid deck. For decks supported on transverse purlins (longitudinal deck span) spaced at 32 inches on center, the equivalent strip width for the aluminum orthotropic deck is 2.2 times that of the steel open grid deck.
- *Flexural Resistance*: The limiting tension or compression stress of the deck in the secondary direction is approximately 33% of that in the primary direction. Aluminum (6063-T6 Alloy) has a different nominal flexural resistance for unwelded material (32.5 ksi) and welded material (10.4 ksi). The resistance of the section is the sum of the unwelded areas times the unwelded resistance and welded areas times the welded resistance. The primary direction has a significantly smaller proportion of area that is welded (15% of the section, which includes the top and bottom plates within 1 inch of the weld) compared to the secondary direction (100% of the section through the welds). The net nominal flexural resistance in the primary direction is 29.2 ksi compared to 10.4 ksi for the secondary direction. A resistance factor of 0.9 is applied to the nominal flexural resistance values.
- *Fatigue Resistance*: The nominal fatigue resistance of the aluminum orthotropic deck in the primary direction is greater than that in the secondary direction. The nominal fatigue resistance of the base metal is classified as Category A (10.2 ksi nominal fatigue resistance) and is equal in both directions. In the primary direction, the base metal at longitudinally loaded FSW joints is classified as Category B (5.4 ksi nominal fatigue resistance). In the secondary direction, the base metal at transversely loaded FSW joints with built in weld backing is classified as Category E (1.8 ksi nominal fatigue resistance). For comparison, the base metal at the welds top of the deck are classified as Category E' (2.6 ksi nominal fatigue resistance).

SECONDARY DIRECTION SECTION PROPERTIES	
PARAMETER	VALUE
Cross Section Area, A	10.13 in2/ft
Moment of Inertia, Ix	51.82 in4/ft
Neutral Axis Ref., ybott	2.450 in
Neutral Axis Ref., ytop	2.550 in
Section Modulus, Sxbott	21.15 in3/ft
Section Modulus, Sxtop	20.32 in3/ft

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

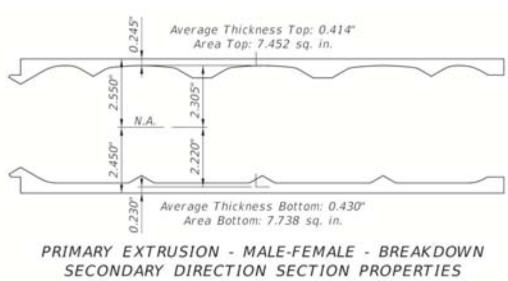

FIGURE 7: Deck Secondary Direction Section Properties

PRIMARY DIRECTION SECTION PROPERTIES	
PARAMETER	VALUE
Cross Section Area, A	14.29 in2/ft
Moment of Inertia, Ix	58.09 in4/ft
Neutral Axis Ref., ybott	2.527 in
Neutral Axis Ref., ytop	2.473 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.

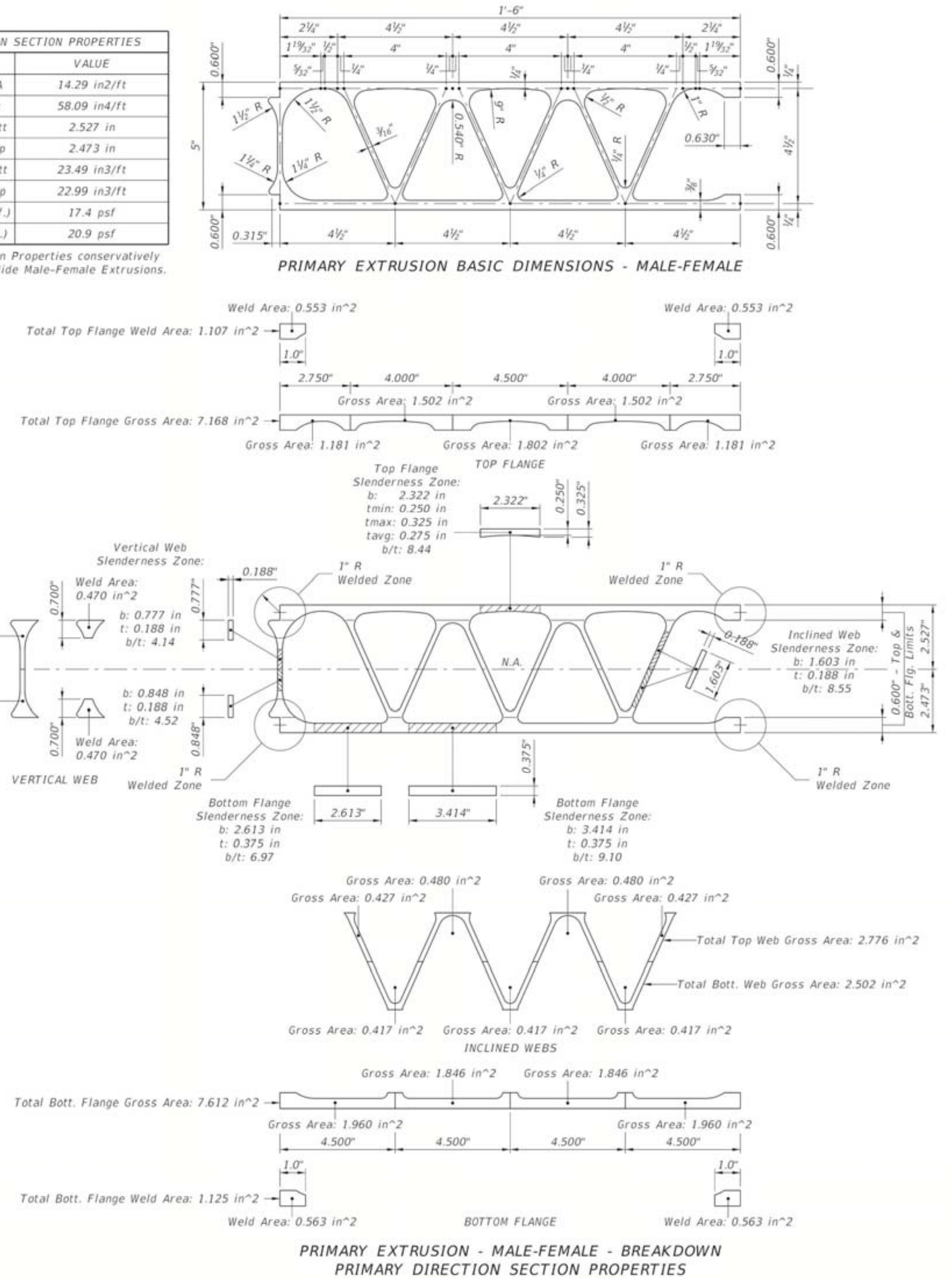

FIGURE 8: Deck Primary Direction Section Properties

Deck Panel Sealed Joints

Sealed transverse joints (transverse and longitudinal) between the deck panels, and longitudinal curbs or steel traffic barriers accommodate fabrication and installation tolerances, permit deck unit sizes manageable for shipping and handling, and accommodate structural deformation and thermal movement. The proposed joints consist of a ¾" to 1" maximum openings between panels filled with continuous joint filler, such as a backer rod with poured low modulus silicone joint sealant, although other joint sealant materials can be used. Structural deformations and differential thermal movements are anticipated to be small (total movement less than ¼").

Bolted Connections

Slip-resistant connections are recommended between parts subject to live loading. Repeated cycles of slip between the deck and steel framing can cause fretting that might wear protective steel coatings. Slip of the connections can be permitted under thermal loading where the number of cycles of slip is low. To resist slip, the aluminum deck is attached to the top flange of the stringers with a bolted connection using fully pre-tensioned, ¾" diameter ASTM A325 high strength bolts or equivalent.

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 Class B surface condition (0.5 coefficient of friction) for ASTM A325 high-strength bolts. 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, such as Carboline Carbozinc 11, with 6.0 mils dry film thickness.

For corrosion resistance, 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 is required between the fastener and aluminum surface to prevent galling.

Because the aluminum deck consists of hollow extrusions, blind type fasteners can also be considered to make the connection between the deck and steel framing, such as ¾" Lindapter High-Clamping Force Hollo-Bolts (LHB-HCF). The LHB-HCF fastener can be field installed. The fastener is installed in 1-3/8" diameter standard size holes (1/16" in diameter larger than the split-sleeve). However, because sustained pre-tensioning proof loads for this type of fastener is significantly lower than similarly sized conventional ASTM A325 bolts (10 kips vs. 28 kips), this bolt type is not recommended as the primary fastener type in a slip-resistant connection subject to cyclic loading.

Wearing/Friction Course

The aluminum deck receives a skid-resistant wearing/friction course 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). 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. Although this is the preferred wearing surface by AlumaBridge, LLC, other wearing surfaces can be used.

FIGURE 9: Wearing/Friction Course

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:

TABLE OF WEARING/FRICTION COURSE EPOXY RESIN PROPERTIES		
PROPERTY	**REQUIREMENT (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:

TABLE OF WEARING/FRICTION COURSE AGGREGATE PROPERTIES	
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/friction course 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 degrees 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 degrees F and 40 – 60% relative humidity for a minimum of 24 hours prior to coating application.

Application: Wearing/friction course 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 degrees 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.

Future Maintenance: The two-coat wearing/friction course 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/friction course 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.

Friction Stir Welded Joints

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.

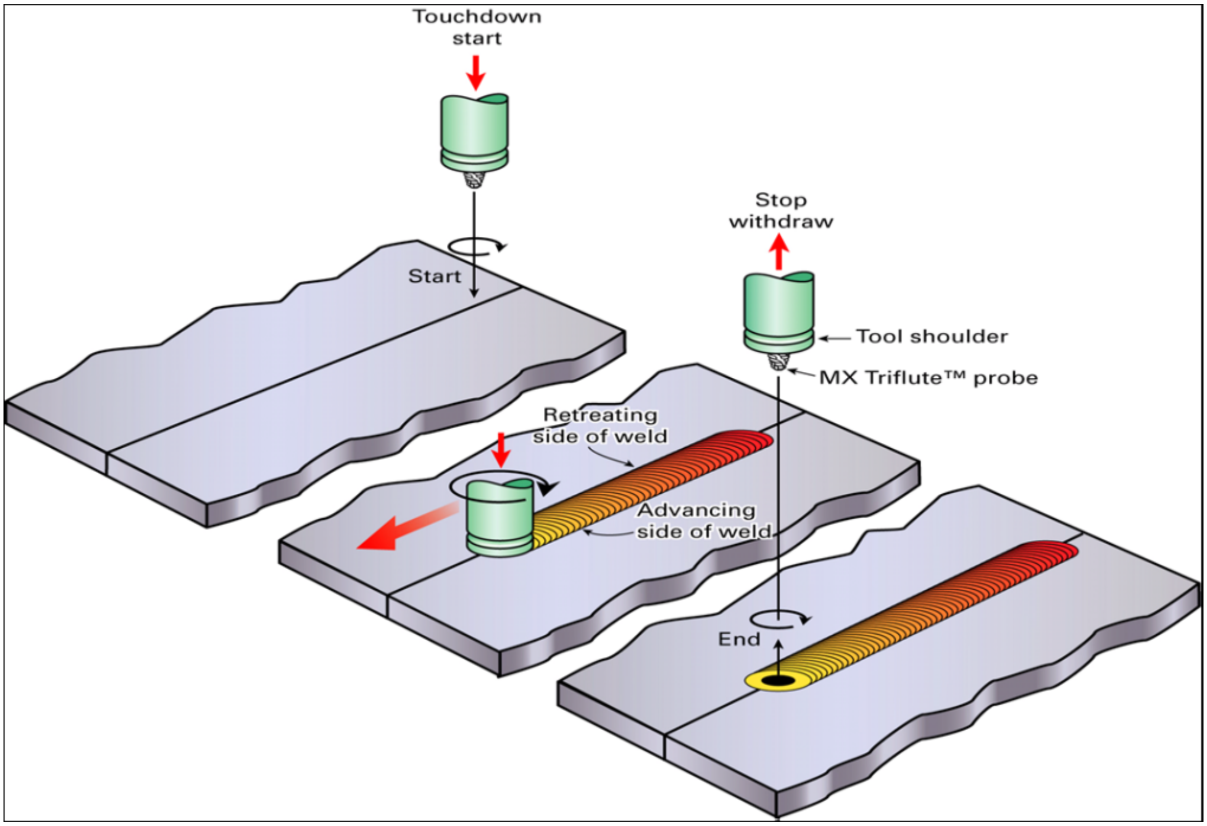

FIGURE 10: Friction Stir Welding Process

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. 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

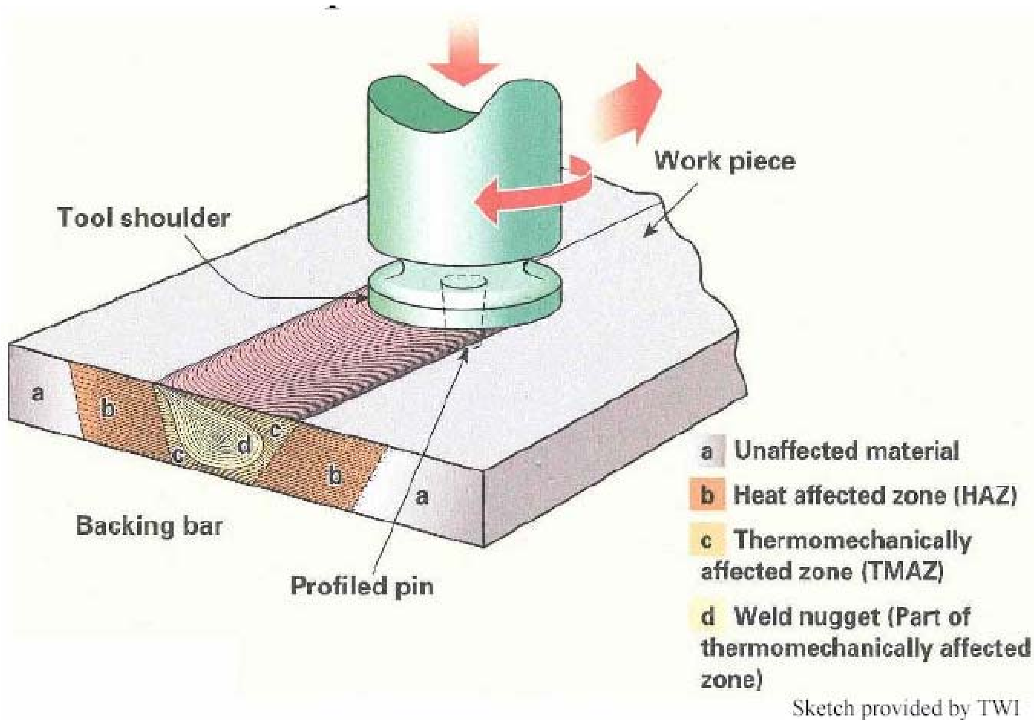

FIGURE 11: Friction Stir Welded Joint

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.

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 or later) and shall be implemented by the deck panel fabricator as follows:

- Aluminum deck is considered as a cyclically loaded, tubular structure 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.

Panel Fabrication Requirements and Tolerances

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:

TABLE OF DECK PANEL TOLERANCES	
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 degrees F. Where temperatures at the time of measurement vary from 70 degrees 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.

Structural Analysis

General

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

System 1 Stresses: 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: The stringers are typically conservatively designed without contribution from the deck panel in composite behavior. However, if a slip critical connection between the deck and stringers is used, the compressive forces in the deck should be considered in the design of the deck.]

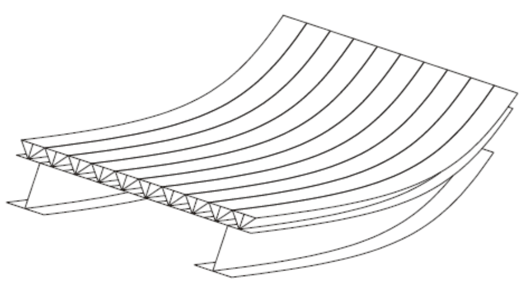

FIGURE 12: Schematic Showing System 1 Flexure

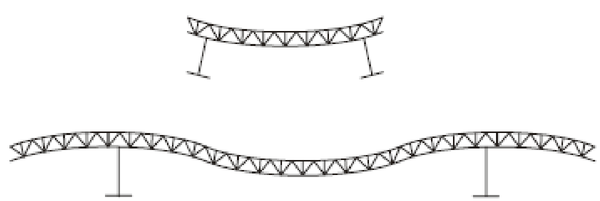

FIGURE 13: Schematic Showing System 2 Flexure

System 2 Stresses: Flexural compression or tension stresses and corresponding deformations in the deck panels introduced as a result of loading of the deck panels between the support members. [NOTE: Panels experience positive and negative flexural stresses at different locations and loading conditions due to continuity of the deck over intermediate supports. Where System 2 Stresses are parallel to the extrusions, finite element analysis (FEA) confirms that 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. Where System 2 Stresses are perpendicular to the extrusions, shear lag effects are not applicable.]

System 3 Stresses: 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. [NOTE: Because the top plate, bottom plate, and inclined webs are integral, the extrusions experience frame action. The System 3 Stresses are complex and not easily computed using manual hand calculations. FEA was performed to compute these stresses. Because the System 3 Stresses do not vary significantly with different deck support conditions, the results of the FEA can be added directly to the other stresses. System 3 Stresses are only added to other stresses where coincident.]

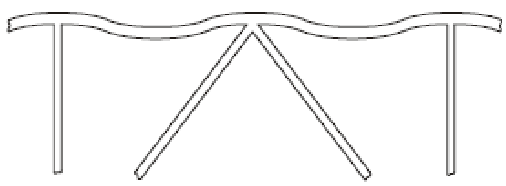

FIGURE 14: Schematic Showing System 2 Flexure

Manual calculations can be used to design the deck system as follows:

- Design/analyze the deck in accordance with *AASHTO LRFD Bridge Design Specifications, 7th Edition* (which is consistent with the *2015 Aluminum Design Manual*) or later.
- Section properties for the deck system in both the longitudinal and transverse directions are computed using CADD or other means (shown above). In the primary direction, the section properties are constant along the length of the panels. The primary direction section properties can be conservatively based on the extrusion with the lowest section modulus (e.g. AlumaBridge Gen II Male-Female Extrusion). In the secondary direction, section properties vary due to the variation in top flange thickness. Transverse section properties should be considered at the following sections:
    - At the minimum section where deck top and bottom flange thickness are a minimum
    - At the minimum section within 1 inch of the FSW welded joints
    - Average section properties for use in stiffness analysis and deflection calculations.
- Stress limits (resistance of elements in flexural compression and flexural tension) are computed in both longitudinal and transverse directions for the Strength I Limit State. For flexural compression, all elements of the deck panels are treated as flat elements supported on both edges. Local buckling resistance (slenderness) of all elements is considered in the flexural compression resistance. [NOTE: All elements have b/t ratios less than λ_1, and thus yielding of elements governs over local buckling.] Stress limits include required deductions for lower welded strength of members. The resistance of the section is the sum of the unwelded areas times the unwelded resistance and welded areas times the welded resistance. The primary direction has a significantly smaller proportion of area that is welded (15% of the section, which includes the top and bottom plates within 1 inch of the weld) compared to the secondary direction (100% of the section through the welds). The net nominal flexural resistance in the primary direction is typically 29.2 ksi compared to 10.4 ksi for the secondary direction. A resistance factor of 0.9 is applied to the nominal flexural resistance values.
- Fatigue resistance is computed for the various fatigue details (see below).
- Transverse System 2 deck system flexural stresses for the deck panel are computed with wheel loads applied between the stringers. Both maximum positive and negative moment intensity is computed using an equivalent strip width for distribution of wheel load. Previous testing has confirmed that the equivalent strip width equations for concrete decks can be applied to the

aluminum deck panels. Shear lag effects which increase deck panel stresses are also considered (see below).

- Deck dead load forces can be neglected in computing System 2 Stresses as dead load is a small fraction (1/500) of the live load.
- Live load deflection is limited to L/800, where L is the deck span length. Wheel loads placed at the free edge of the panel control live load deflection. In the Florida DOT research, FEA demonstrated that live load deflection was below the L/800 limit with a stringer spacing of 6'-0". Calculations conservatively considered the deck as supported in a simple span condition (continuity not considered) using only the stiffness of the deck panel. These calculations can be omitted if a stringer spacing of 6'-0" or less is used and with the deck supported on at least three stringers.

Strength and Service Limit States

Loading Magnitudes: The deck panel performance is analyzed and evaluated in accordance with AASHTO LRFD Articles 7.5.1 thru 7.5.3. Static loading is 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 Limit States.

The deck system was evaluated for AASHTO LRFD HL-93 Design Truck and Design Tandem wheel loads. Based on FEA, the results from AASHTO LRFD HL-93 Design Truck are similar to the Design Tandem, and thus it is not necessary to compute load effects for both Design Truck and Design Tandem. This is similar to the performance of concrete decks.

Both a single lane of traffic and two lanes of traffic are considered with corresponding AASHTO LRFD Multiple Presence Factors, *m*. For longitudinal System 2 Stresses, a single wheel line (Multiple Presence Factor, m = 1.20) produces a greater moment intensity for both positive and negative moments than two lanes of traffic. For transverse System 2 Stresses, a single wheel line (Multiple Presence Factor, m = 1.20) produces a greater moment intensity for positive moment and two wheel lines from different lanes (Multiple Presence Factor, m = 1.00) with adjacent wheel lines spaced 4'-0" apart produces a greater moment intensity for negative moment.

- Wheel loads are applied to deck top surface as 20" (transverse) x 10" (longitudinal) patch load.
- Wheel loads are applied at the loading levels below.

TABLE OF WHEEL PATCH LOADS (kips) (1)			
AASHTO LRFD LOAD CASE	**LOAD FACTOR, Υ_{LL}**	**HL-93 DESIGN TRUCK SINGLE-LANE (2)**	**HL-93 DESIGN TRUCK TWO-LANE (3)**
SERVICE I	1.00	25.54	21.28
SERVICE II	1.30	33.20	27.66
STRENGTH I	1.75	44.69	37.24

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 Truck 32-kip Rear Axle (Q=16-kip Wheel), magnified for Dynamic Load Allowance (IM=0.33) and Two-Lane Multi-Presence Factor (m=1.00).

Shear Lag Effects: Because stresses are distributed throughout the deck by way of the inclined webs, System 2 Stresses in the primary direction vary across the width of the top and bottom plates 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 FEA stress contours as "ripples". The magnitude of the shear lag effects was evaluated by analyzing the variation in stress within the top and bottom plates. In the Florida DOT research, 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. The shear lag multiplier for the top plate is 1.20 and for the bottom plate is 1.09. 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, such as those computed using equivalent strip method, need to be magnified by the shear lag multipliers to yield accurate results.

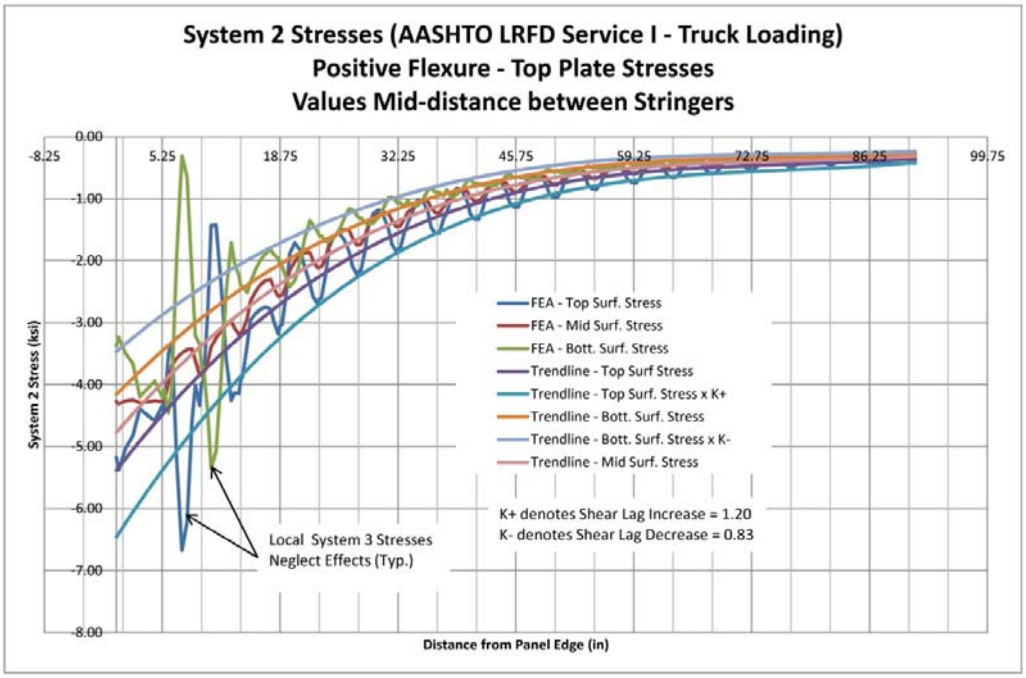

FIGURE 15: Top Plate Longitudinal Distribution Showing Shear Lag Effects

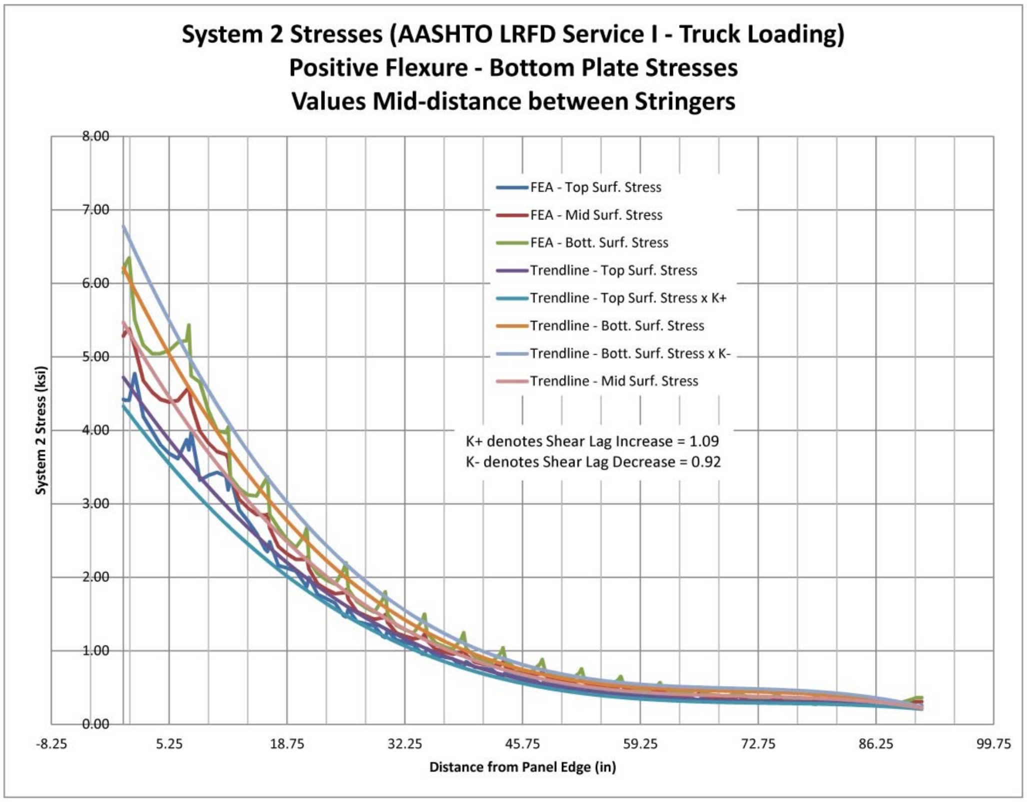

FIGURE 16: Bottom Plate Longitudinal Distribution Showing Shear Lag Effects

Local System 3 Effects: Wheel patch loading on the top plate introduces flexural stresses (System 3 Stresses) in the top plate. Although the magnitude of stress in the secondary direction is larger, from approximately one-way bending of the top plate between the inclined webs, there are corresponding smaller stresses in the primary direction due to biaxial plate bending. The System 3 Stresses are directly additive to System 1 Stresses and directly additive to System 2 Flexural Stresses, where coincident. For example, System 2 Stresses from maximum positive flexure, are typically coincident with the System 3 Stresses as the maximum positive flexure usually occurs directly below the point of the load. The magnitude of the System 3 Stresses from FEA are as follows:

TABLE OF STRENGTH I SYSTEM 3 FLEXURAL STRESSES	
TOP PLATE PRIMARY DIRECTION	**TOP PLATE SECONDARY DIRECTION**
1.9 ksi	5.6 ksi

The above stresses are to be added, when stresses are computed using simple closed-form equations, such as those computed using equivalent strip method.

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 DOT, 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 4.6.2.1.4(c) 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):	$b_e = 26.0 + 6.6S$
Interior Strip Width (Negative Flexure):	$b_e = 48.0 + 3.0S$
Transverse Edge Width (Positive Flexure):	$b_e = X + 0.5*(26.0 + 6.6S)$
Transverse Edge Width (Negative Flexure):	$b_e = X + 0.5*(48.0 + 3.0S)$

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

Fatigue Limit State

Fatigue Loading: The deck panel performance is analyzed and evaluated for fatigue in accordance with AASHTO LRFD Article 7.6.1. Fatigue loading is 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. 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.

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
$(\Delta F)_N$ denotes Nominal Fatigue Resistance (see below.)

In order to evaluate the performance of specific fatigue sensitive details, the details are 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:

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

Υ_{LL} denotes the Fatigue I Limit State load factor
Q denotes the fatigue truck wheel load
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:

- *Base Metal*: Category A [NOTE: Applies to base metal throughout the deck panels loaded for System 1, 2 or 3.]
- *Welded Joint (Stresses Loaded Normal to Weld Axis)*: Category C or Category E [NOTE: This detail applies to the welded joint in the top plate subject to System 3 Flexural Stresses and top and bottom plates in the secondary direction subject to System 2 Flexural 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 and with weld backing to remain. Category C applies to the top surface of the deck at the welded joint. 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 than FSW joints. 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 is conservatively classified as Category C. Category E applies to the unfused seam at the reentrant corner of the single-sided friction stir welded joint built-in weld backing. This fatigue detail is not classified by AASHTO. The reentrant corner at the FSW joint seats introduces stress raisers that could be fatigue concern. 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. See further discussion below.]

- *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. The bolted connection of the deck to stringers typically occurs at the location of negative moment where the bottom of the deck remains in compression under all fatigue loading scenarios and thus is not subject to tension or stress reversal.]
- *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 or System 3 Stresses in the primary direction. 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.]

Stress Range and Number of Fatigue Cycles: 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:

$(\Delta F)_N = C_f^{\ -1/m} = (\Delta F)_{TH}$ where, (AASHTO LRFD Equation 7.6.2.5-2)

$(\Delta F)_N$ denotes Nominal Fatigue Resistance,
C_f denotes the x-intercept of the logarithmic S-N curves,
m denotes the slope 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.

TABLE OF INFINITE FATIGUE VALUES				
	Category A	**Category B**	**Category C**	**Category E**
Design $(\Delta F)_N = (\Delta F)_{TH}$	10.2 ksi	5.4 ksi	4.0 ksi	1.8 ksi
N at $(\Delta F)_{TH}$	$5x10^6$	$5x10^6$	$5x10^6$	$5x10^6$

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 7.6.2.5-3)

N denotes the Load Cycles per Truck Passage (AASHTO LRFD Equation 7.6.2.5-3)
$(ADTT)_{SL}$ denotes single-lane ADTT (AASHTO LRFD 3.6.1.4)

System 2 Fatigue Stress Range: System 2 Fatigue Stress Range values are computed using manual calculations similar to that for the System 2 Strength I Flexural Stresses. All calculated values should be below the AASHTO LRFD Nominal Fatigue Resistance values.

System 3 Fatigue Stress Range: System 3 Fatigue Stress Range values were computed using a three-dimensional solid element model and included moving fatigue loading, thermal stress range, and braking loads. The System 3 fatigue analyses considered 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 well below the Constant Amplitude Fatigue Threshold, $(\Delta F)_{TH}$, value of 10.2 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 0.63 ksi, which is well below the Constant Amplitude Fatigue Threshold, $(\Delta F)_{TH}$, of 4.00 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 (3/16" 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 3.25 ksi, which is well below the Constant Amplitude Fatigue Threshold, $(\Delta F)_{TH}$, of 10.2 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*: 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 0.63 ksi and 1.25 ksi, which is below the Constant Amplitude Fatigue Threshold, $(\Delta F)_{TH}$, of 1.8 ksi for Category E, which is the fatigue detail with the lowest fatigue limit. 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. Because one-half of the FSW joint is solid and a fully integral part of the extrusion, the risk is low that the horizontal feature will propagate into the solid half of the joint. Research on similar weld details of similar friction stir welded aluminum alloys has demonstrated that the friction stir welding produces residual compressive stresses across the weld seam that inhibits crack propagation.

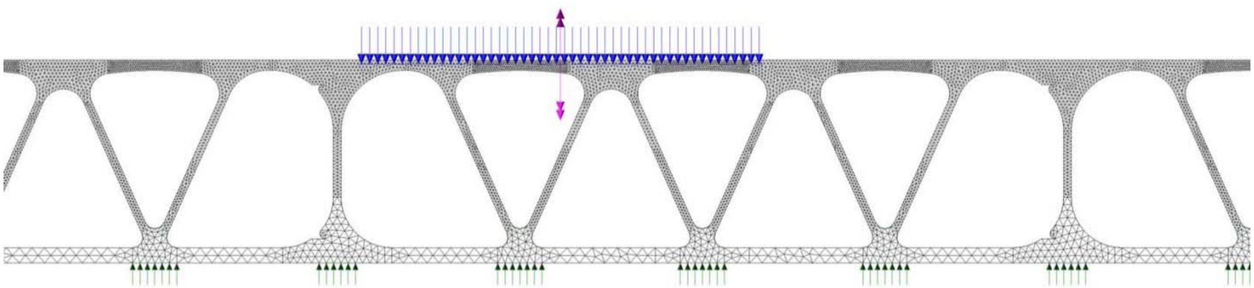

FIGURE 17: System 3 Model Close-up

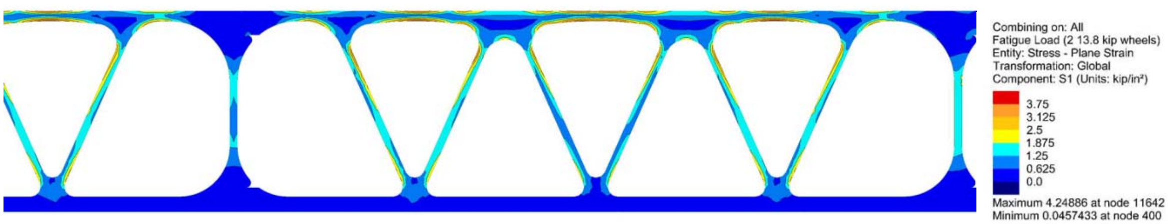

FIGURE 18: System 3 Fatigue Analysis Results

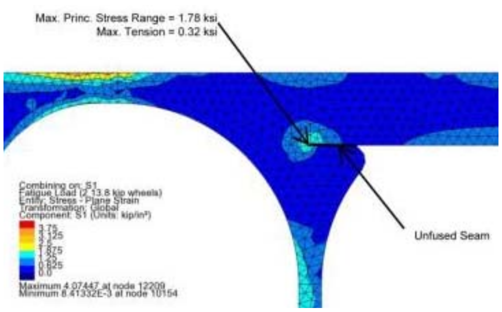

FIGURE 19: System 3 Fatigue Analysis Results

Corrosion

General/Atmospheric Corrosion

Aluminum 6063-T6 Alloy has excellent inherent corrosion resistance. The material produces a tightly bonded aluminum oxide film that is generally impermeable and not easily displaced by running water. When atmospheric corrosion occurs, the aluminum experiences only superficial pitting with a maximum depth that is a fraction of the material thickness. Pitting is primarily an aesthetic problem that generally does not affect the strength of the aluminum.

Atmospheric corrosion can be mitigated by avoiding standing water. Wetness from condensation generally exists for only a limited duration and is generally not a concern. Stormwater runoff will typically be controlled with sealed joints and strategically located drains. Stormwater that passes the sealed joints can be mitigated by sealing the ends of the panels and/or providing drain holes in the bottom of the deck panels. Generally, the solid deck will minimize the presence of an electrolyte needed for corrosion development.

Aluminum is commonly used for miscellaneous structural elements on bridges and along roadway facilities (e.g. railings, sidewalks, light poles, sign structures, etc.) and are commonly found in extremely aggressive marine environments and where deicing salts are used. Where properly detailed, these aluminum components and corresponding fasteners have a track record of successful performance.

Galvanic Corrosion

With dissimilar metals, the less noble (anodic) material sacrifices itself to protect the more noble (cathodic) material. The aluminum alloy deck is supported on low alloy or mild (carbon) steel framing that typically contains zinc coatings (e.g. inorganic or organic primers, hot dip galvanized, or metalized coatings). The deck is typically connected using hot-dip or mechanically galvanized (zinc coated) fasteners. Stainless steel (Type 316) fasteners are also sometimes used with aluminum.

Per the galvanic series, from least noble to most noble:

TABLE OF ELECTRICAL POTENTIAL (GALVANIC SERIES)	
Material	**Electrode Potential (volts)**
Zinc Coating	-1.00 to -1.08
Aluminum Alloy	-0.76 to -1.00
Mild (Carbon) Steel	-0.58 to -0.72
Low Alloy Steel	-0.56 to -0.64
Stainless Steel (Type 316)	0.00 to -0.10

As such, the zinc coating on the steel framing and fasteners sacrifices itself to protect the aluminum and steel. Because the zinc material will eventually be exhausted from this process, it is recommended to maximize the amount of zinc in the coatings. Once the zinc is exhausted, the aluminum alloy will be the least noble material and will sacrifice itself to protect the more noble low alloy steel or carbon steel.

The rate of corrosion is dependent on the magnitude of the potential difference and the ratio of the surface areas of the dissimilar metals. Aluminum is generally close to mild steel and somewhat close to low alloy steel in the galvanic series and thus the potential difference is reasonably low. On the other hand, there is a fairly significant potential difference between aluminum and stainless steel. In addition, the greater the area of the less noble (anodic) component relative to the more noble (cathodic) component the slower the rate of corrosion (i.e. the lower the current density). The surface area of the anodic aluminum deck is typically greater than the surface area of the cathodic steel stringer and far greater than that of the cathodic steel fasteners. With limited stormwater runoff present at the connections, the rate of corrosion of the aluminum deck will be slow once the zinc is exhausted.

A high number of cycles of loading that produces slip in connections between the aluminum deck and steel framing can cause fretting that will wear the protective coatings. In this event, slip-critical connections are recommended. Where a slip-critical connection is not practical, physical separation between the aluminum and steel is recommended such as a bituminous coating or resilient pad. Fully tensioned high-strength bolts are not compatible with resilient pads or bituminous coatings as compression of the pads or thicker coating will result in creep that will reduce the pretension force. Slip-critical connections with aluminum are certified for only a limited number of coating systems. Hot dip galvanized steel and inorganic zinc primer in contact with abrasion blasted aluminum are both classified as Class B surface conditions. If fully tensioned high-strength bolts are not used, and resilient pads or bituminous coating is used, with or without bolts with polymeric sleeves and washers to provide electrical separation, a means to secure the nuts from loosening due to vibrations is recommended.

Composite Behavior and Thermal Effects

Where the deck is made composite with the supporting member, the fully effective width is typically the tributary width of the supporting member (i.e. spacing between the supporting members) for an interior member or half of the tributary width plus the cantilever overhang for an exterior member. The deck panel is typically discontinuous at the end of the span with deck joints and is effectively non-composite at this location. The effective width of deck that is composite increases away from the joints until the full effective width is achieved, which yields variations in composite section properties along the length of the support members.

Where the aluminum deck is oriented with the primary direction perpendicular to the axis of the supporting member, an additional shear lag effect for the top plate must be considered. Although the deck bottom plate is connected directly to the top flange of the supporting member, the deck top plate is only connected to the bottom plate by a series of inclined and vertical web members. A finite-element analysis confirmed that the deck bottom plate effective width increases linearly from the deck joints at a 45 degree angle on each side of the axis of the supporting member, until the full effective width is achieved. However, the top plate effective width is different than the bottom plate due to shear lag effects caused by the deformation of the deck trussed panels. The effective width of the top plate at a given section is approximately equal to the square of the ratio of the bottom plate effective width at that section to the fully effective width. Where the aluminum deck is oriented with the primary direction parallel to the axis of the supporting member, the effective width of the top and bottom plate are essentially equal and both increase linearly from the deck joints. Where the aluminum deck is supported on transverse purlins on stringers, the flexibility of the purlin webs relieve much of the composite action and reduce the axial force in the deck.

Shear flow between the deck and supporting members, used to design the connection between the deck and support members, can be computed using manual calculations. However, because the section properties are variable along the length of the support members and there are sometimes deck joints at intermediate points along the length of the member that introduce a discontinuity, simple VQ/I calculations to compute shear flow are not applicable. Shear flow must be determined by computing the rate in change (i.e. slope) of the deck compressive force along the length of the supporting members. 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.) Where there is discontinuity in the deck panel at an intermediate point, the decrease in composite section and increase in bending moment approaching the discontinuity results in a rapid rate of change in deck compression with a corresponding spike in shear flow each side of the discontinuity.

Aluminum has a different coefficient of thermal expansion than steel (12.8×10^{-6} /deg. F for aluminum vs. 6.5×10^{-6}/deg. F for steel.) As such, thermal movement and/or forces from restraint of thermal movements must be considered in the design of the deck system and supporting steel framing.

Where a slip-critical connection is used to connect the aluminum deck to steel framing or in steel framing end connections where the aluminum deck is composite with the steel framing, both live load and thermal effects contribute to the forces acting to cause slip. Although it is undesirable to permit slip under a large number of cycles of live load, as this can cause fretting that may wear the steel coatings, it is not necessary to also design the connection for the restraint forces corresponding to full design temperature range. Slip from a low number of cycles of temperature variation can be tolerated without similar concerns of fretting. Full design temperature range, in the magnitude of 90 to 135 degrees F, typically occurs over a longer seasonal time frame, while smaller temperature variations, in the range of 18 to 27 degrees F, occur daily. If the slip-critical connection is designed to resist live load plus a thermal restraint force corresponding to that from the diurnal temperature range (i.e. change in temperature from daytime high to nighttime low) slip is anticipated to occur on an infrequent basis. When the connection slips, slip movement relieves the thermal restraint forces, and there needs to be another build-up of thermal restraint forces before slip can occur again. The magnitude of the number of cycles of thermal loading is a fraction of that of live loading ($5x10^4$ daily thermal cycles vs. $5x10^7$ design truck live load cycles) over a 75 year service life. As such, thermal slip is not anticipated to be a significant factor in fretting of steel coatings. It is unclear whether limited slip will produce a "polishing" effect that will reduce the coefficient of friction of the faying surfaces and corresponding slip resistance.

Where the aluminum deck is made composite with the steel members, the relative thermal movement between the deck and steel member, and the eccentricity of the deck to the steel member results in flexural deformation of the composite section (i.e. the composite deck and steel members will deflect upwards (temperature increase) or downwards (temperature decrease) between the supports with corresponding rotations at the end connections). Where the end connections provide little or no restraint to end rotations (e.g. simple span stringers with discontinuous deck panels over the end connections and where the stringers are supported on top of the floorbeams or where the stringers frame into a torsionally flexible end floorbeam), then the deformations will occur without consequence. Where the end connections provide end restraint (e.g. simple span stringers where the stringers frame into intermediate floorbeams and the stringer on the opposite side of the floorbeam opposes the end rotations), a resisting end moment is generated and must be considered in the end connection design.

Deck Cross Slope and Drainage

The aluminum deck with a solid surface can be configured with or without a transverse cross slope, including straight and parabolic crowned sections, to match the existing cross slope condition. On many movable bridges, the existing deck has no transverse cross slope on the movable span and the deck transitions to a crowned section off of the movable span. Most bridges are located along a roadway vertical profile or grade that promotes longitudinal drainage and directs the water to the portion of deck that contains a cross slope. In addition, there typically is a transverse open joint between the movable span and fixed approach spans. In order to minimize the water that enters the transverse open joints, introduction of joint seals into the joint assemblies should be considered. Alternatively, gutters below the open joints that collect and redirect the water can be considered.

Where the deck includes transverse cross slope, deck drainage can be addressed with the introduction of drains through the deck. A series of holes can be drilled through the deck and aluminum pipes inserted into the holes and welded to the deck. The top of the pipe will typically extend a ¼" above the top of the deck so that it will be flush with the wearing/friction course. The bottom of the pipe can extend below the bottom of the deck a sufficient length to install a pipe coupling and pipe extension to direct water past the steel framing. The number and size of the drains are a function of the stormwater runoff requirements and permitted spread. It is also recommended to seal the longitudinal joint between the deck and curb or barrier at the gutter line.

Deck panels are fabricated flat. In order to accommodate a curved transverse parabolic cross slope or longitudinal vertical profile, the deck can be deformed to the curved shape by loading the panel. In most cases the mid-ordinate value of the curve is relatively small and can be accommodated with reasonable loading. The fasteners use to secure the deformed panel to the steel framing must be sized accounting for the hold down force including prying action as required. In addition, the deformation introduces locked-in flexural stresses in the deck that must be added to the other computed deck panel stresses.

Traffic Railings

Traffic railings are required to be crash-tested per MASH or NCHRP Report 350 requirements. These requirements specify that the traffic railing system be crash-tested attached to the foundation or deck that the railing will be normally attached. To date, no crash testing of traffic railings on the aluminum orthotropic deck has been performed. Because it is not practical to crash test traffic railings on various deck systems, FHWA routinely will approve the mounting of traffic railings on alternative deck systems, where it can be demonstrated that the deck system has equivalent strength and stiffness to resist the traffic railing impact forces. Traffic railings can be through bolted to the aluminum orthotropic deck with backing plates on the underside of the deck. Calculations have been performed that demonstrate that the 5-inch aluminum orthotropic deck has greater strength and stiffness and better load distribution properties than corresponding 5-inch steel open grid deck. As such, if a crash tested traffic railing is mounted directly to steel open grid deck on an existing bridge, the traffic railing can also be mounted to an aluminum deck. Furthermore, calculations have been performed that aluminum orthotropic deck has sufficient capacity to resist NCHRP Report 350 Test Level 4 (TL-4) traffic railing impact forces for typical steel post and beam traffic railings.

Deck and Steel Framing Configurations

The aluminum orthotropic deck is adaptable to different bridge, cross section, and steel framing configurations. The deck system can also be configured for accelerated bridge construction (ABC). Below are several example that illustrate the adaptability and ABC capability.

Example 1

The following proposed configuration was developed to replace 5-inch deep steel open grid deck and steel stringers on a four-lane double-leaf bascule bridge commonly found in Florida (e.g. Las Olas Boulevard Bridge, Ft. Lauderdale, Florida).

Existing Deck and Support Framing: The existing deck and steel framing on the bascule span are as follows:

- The bridge cross section consists of two 23’-0” wide clear roadways separated by a 2’-0” wide traffic separator and bordered by steel curbs. The deck is flat (no cross slope) in the transverse direction and follows a vertical profile (500’ long vertical curve with 6% approach grades) in the longitudinal direction.
- The deck system is a 5-inch 4-way (diagonal) steel open grid deck. The deck system transitions to a 3-inch deep concrete filled steel grid deck over the bascule piers.
- The deck is supported on and welded to 14-inch deep steel stringers spaced at 4’-2½” on center and spacer bars welded to the top of the main girders.
- The stringers span longitudinally between 36-inch deep rolled steel floorbeams and a 106-inch deep riveted built-up steel floorbeam at the front of the bascule pier spaced at 18’-6” on center. The stringers frame into the webs of the floorbeams using connection angles.
- The floorbeams span between riveted built-up steel main girders spaced at 46’-5” on center.

- The steel curbs are supported on cantilevered brackets outboard the main girder. The steel traffic separator is supported on and welded to the top of the steel open grid deck.
- Although the steel grid deck is welded to the stringers and main girders, it does not contribute to the strength and stiffness of the stringers and main girders.

Proposed Deck Modifications: The aluminum orthotropic deck system and required bridge modifications to implement the deck system are as follows:

- Similar to the existing steel open grid deck, the aluminum orthotropic deck panels will be supported directly on top of new stringers with the panels oriented with the extrusions (primary direction) oriented perpendicular to the bridge longitudinal axis.
- The deck will be replaced in two phases with half of the roadway width replaced in each phase. The existing traffic separator must be removed before the existing deck can be removed. The existing steel open grid deck and stringers will be removed in each floorbeam bay by flame cutting the stringers and deck at the floorbeams and main girders. The spacer bars on the top of the floorbeams and main girders are removed by careful grinding. The existing deck and stringers can be removed as a single unit.
- The new deck system will consist of ten deck panels each measuring 9'-2" wide x 23'-10" long and two deck panels each measuring 10'-1¾" x 23'-10" long. The 9'-2" wide panels each consist of one (1) 1'-6" male-male extrusion, three (3) 1'-6" male-female extrusions, two (2) 1'-1½" male-female extrusions, and two (2) 1'-1½" end extrusions trimmed 1¼" to a 1'-0¼" width. The 10'-1½" wide panels each consist of one (1) 1'-6" male-male extrusion, three (3) 1'-6" male-female extrusions, one (1) 1'-1½" male-female extrusion, and two (2) 1'-1½" end extrusions trimmed 2" to an 11½" width.
- 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, especially in consideration of the significant number of fasteners required to connect the deck to the stringers. New stringers are bolted to the deck in the shop. The ability of the deck to span greater distances than steel open grid deck permits the stringers to be re-spaced. The stringers can be spaced in a configuration that avoids support of the deck on the main girders where connection is more difficult, especially where the main girders are of riveted built-up construction. The configuration also avoids support of the deck on the existing floorbeams that would otherwise require field bolting to the hollow extrusions. The new stringer spacing varies between 3'-11" and 5'-9½", which is less than the maximum recommended spacing of 6'-0". The deck cantilever overhang varies from 1'-2" at the center of the bridge and 1'-9" at the curbs, which is less than the maximum recommended cantilever of 2'-0". The variable spacing is used to clear existing stringer spacing and coordinate with the existing riveted built-up floorbeam web stiffener locations. At the end of each floorbeam bay, the deck cantilevers longitudinally 9" over the floorbeams. The inclined or vertical deck web members are located at the ends of the stringer flanges. New stub stringers are provided in line with the other stringers on the tip side of the end floorbeams.
- The length of the stringers are established to permit the stringers to clear the floorbeam top flange during panel installation. New structural tees are bolted to the web of the floorbeam with the stem of the tee aligned with the stringer web. Because the new stringers are offset from the existing stringers, the holes in the floorbeam web for the tees can be drilled prior to removing the existing deck and stringers. The stringers are bolted to the tee stems with connection plates on each side of the web. Pre-drilled slotted holes in the tee flanges and stringer webs accommodate fabrication and erection tolerances.
- The new stringers, connection plates and new floorbeam connection tees are hot-dip galvanized.

- The deck is bolted to the new stringers with a series of fully pre-tensioned, conventional ¾” diameter tension control type high-strength bolts in a slip-critical connection. Bolts, nuts and washers are mechanically galvanized. For each stringer there are two lines of bolts spaced at 4½” or 9” on center depending on the computed shear flow. Bolts are inserted in standard oversize holes in both the deck plate and stringer flange. Because aluminum deck panels are hollow, special tools are used to deliver the bolts down the interior of the extrusions and install them in the holes. With tension control type bolts, the bolt and nut are held from the exterior and so there is no need to separately secure bolt head on the interior during tensioning.
- The deck system includes transverse expansion joints between the panels with the joints located over the floorbeams and midway between the floorbeams (i.e. near the stringer midspan). Longitudinal expansion joints are also provided between the two sets of panels at the center of the bridge below the traffic separator and between the sets of panels and the steel curbs. The expansion joints consist of a 1” openings between the panels filled with continuous joint filler. The joint filler is a Watson Bowman, Wabo Evazote Seal (1¼” wide x 2” deep).
- A new aluminum traffic separator weldment is bolted to galvanized steel angles through bolted to the top of the deck panels. In order to safely maintain traffic with phased construction, a temporary traffic railing is bolted to the top of the deck along the drop off hazard created when the existing deck system is removed and before the new deck system is installed. The holes through the deck for the traffic separator are used for bolting the temporary traffic railing to the deck panels.

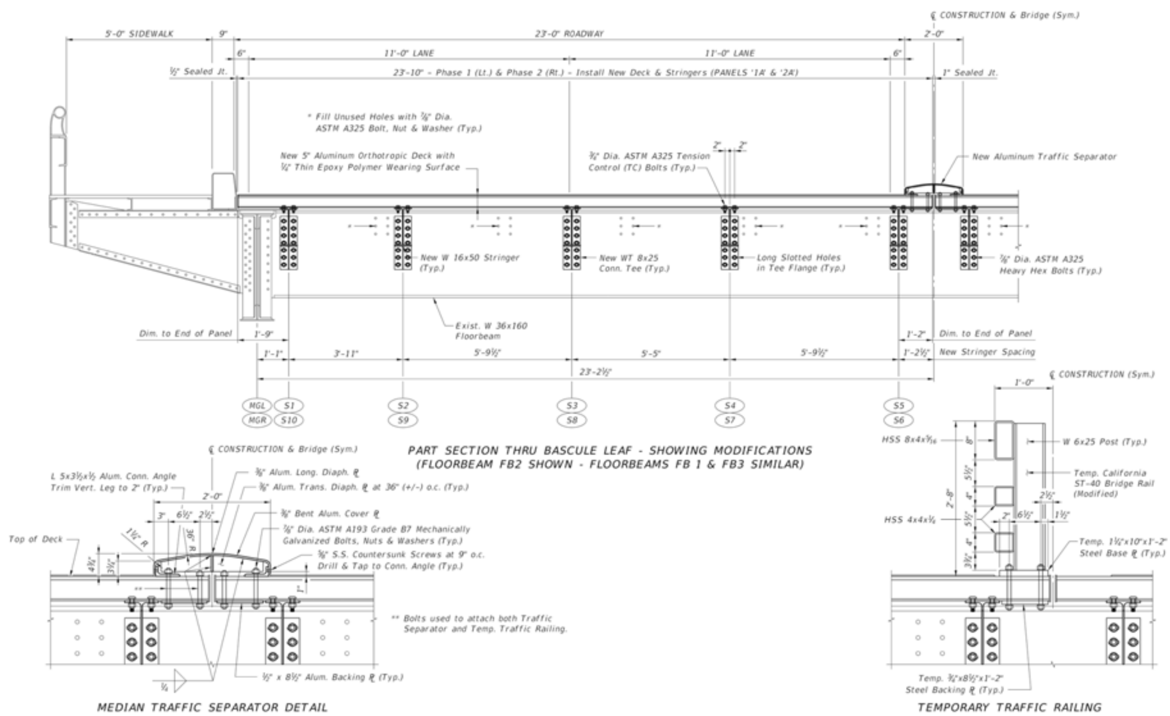

FIGURE 20: Example 1 – Part Transverse Section

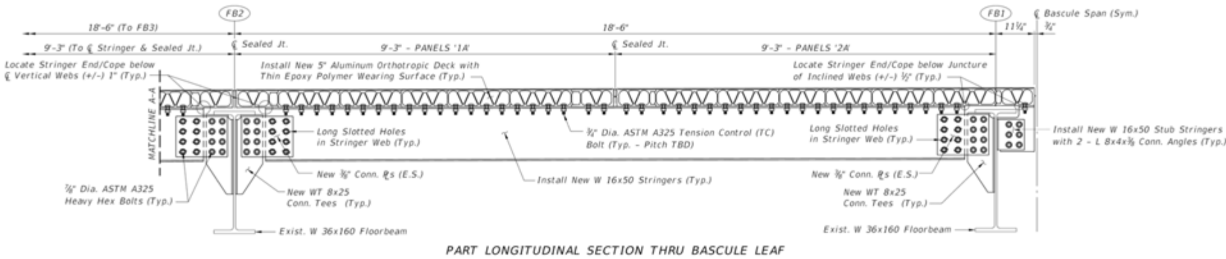

FIGURE 21: Example 1 – Part Longitudinal Section

Example 2

The following proposed configuration was developed to replace 4-inch deep steel open grid deck and 7-inch deep steel purlins on a four-lane vertical-lift bridge (Marine Parkway Bridge, Bronx-Queens, New York).

Existing Deck and Support Framing: The existing deck and steel framing on the lift-span, tower spans, and through-truss spans are as follows:

- The bridge cross section consists of two 24'-0" wide clear roadways separated by an aluminum median barrier and bordered by an aluminum traffic barrier along the west truss and steel traffic barrier along the east truss. The deck has a parabolic crown with a 3" rise and follows a vertical profile (2583.5' long vertical curve with 3.54% approach grades) in the longitudinal direction.
- The deck system is a 4-inch deep rectangular steel open grid deck supported on a series of 7-inch deep transverse steel purlin channels spaced at 32" on center.
- The purlins bear on and span transversely across a series of 27-inch deep steel stringers spaced at 6'-2" on center.
- The purlins for each half of the roadway are offset from each other and overlap at a stringer at the center of the bridge to facilitate phased construction when the steel open grid deck was installed.
- The stringers span longitudinally between 72-inch deep floorbeams of riveted built-up construction spaced at 38'-7" on center.
- The floorbeams span between through trusses spaced at 55'-0" on center.
- The traffic barriers are supported on purlins that extend beyond the edges of the grid deck. The median barrier is supported on the top of the deck.
- Although the steel grid deck is welded to the purlins, it does not contribute to the strength and stiffness of the purlins and although the steel purlins are bolted to the stringers, the deck and purlins do not contribute to the strength and stiffness of the stringers.

Proposed Deck Modifications: The aluminum orthotropic deck system and required bridge modifications to implement the deck system are as follows:

- The deck will be replaced in two stages with half of the roadway replaced in each stage.
- The existing median barrier and traffic barriers will need to be temporarily removed before the existing deck can be removed.
- The existing steel open grid deck and associated steel purlins will be removed the length of each floorbeam bay by unbolting the steel purlins from the stringers. The existing deck and purlin panels can be removed as three individual units.
- After the existing deck panels are removed, the portion of existing purlins that overlap with the first half of the new deck, must be coped to provide clearance for the 5-inch aluminum orthotropic deck, which is 1" deeper than the existing 4-inch steel open grid deck.

- In order to facilitate reuse of the existing traffic barriers, which include posts that are welded to the skin plates and do not include a means of adjustment, the purlins for the new deck system must be accurately located at the same location as the existing purlins. The existing purlin spacing will be be field verified using accurate measurement techniques.
- The new deck system for the first half of the bridge will consist of two deck units each measuring 12'-9" wide x 38'-6" long. Each panel will consist of a series of extrusions including: one (1) 1'-6" wide male-male extrusion, six (6) 1'-6" male-female extrusions, and two (2) 1'-1½"wide end extrusions. The new deck system for the second half of the bridge will consist of two deck units with the outboard panel measuring 12'-9" wide x 38'-6" long, similar to the two panels of the first half, and the inboard panel measuring 11'-9½" x 38'-6". This panel will consist of a series of extrusions including: one (1) 1'-6" wide male-male extrusion, four (4) 1'-6" male-female extrusions, (2) 1'-1½" male-female extrusions, and two (2) 1'-1½"wide end extrusions that are trimmed 1¼" to a 1'-0¼" width. The deck panels will be oriented with the extrusions (primary direction) oriented parallel to the bridge longitudinal axis. This permits the panels to extend the length of a floorbeam bay without intermediate transverse joints. A series of 6" deep ASTM B221 6061-T6 Aluminum Alloy tees, cut from 12" deep x 15.50 lbs/ft I-beams, will be welded to the underside of the deck panels using continuous metal inert gas (MIG) fillet welds. The purlin tees will extend across the width of the deck panels, oriented perpendicular to the bridge longitudinal axis, and placed at locations matching the existing purlins. On the outboard units, the purlins will extend beyond the outboard edge of the deck panels to support the traffic barrier. The total depth of the deck system (5-inch aluminum orthotropic deck and 6-inch tee purlins) will match the depth of the existing 11-inch deck system (4-inch steel open grid deck and 7-inch channel purlins). To seal the open ends of the deck panels, ½" thick x 5 ½" deep end plates will be welded to each end of the panels with across the full width using MIG fillet welds. The plates will extend ¼" above and below the top surface of the deck.
- A ¼-inch thick two-layer epoxy polymer wearing/friction course will be shop applied to the panels after fabrication including purlin and end plate welding. The wearing/friction course will extend the full length and width of the deck panels except for a short width directly below the median barrier to permit the barrier toe clips to bear directly on the top plate of the deck. The wearing/friction course will be contained along the deck panel edges by the ¼" lip on the end extrusions and at the ends of the panels by the end plates.
- The unit weight of the new deck system (24.5 psf including aluminum deck panels, wearing/friction course, and aluminum purlin tees) has approximately the same unit weight as the existing deck system (24.1 psf including steel open grid deck and steel purlin channels).
- The new deck system will be shipped and installed as four separate longitudinal units (two for each stage to facilitate ABC. The deck for one floorbeam bay and half of the bridge width can be replaced in an overnight period with traffic restored during daytime hours.
- Sealed longitudinal joints, with a width of ¾" to 1", will be provided between the units and between the outboard unit and steel traffic barrier to accommodate fabrication and installation tolerances. Alternatively, the longitudinal joint between the panels can be seal welded with a MIG welded partial penetration groove weld.
- The friction stir welding equipment requires that the deck panels be fabricated flat. The aluminum purlins will be welded to the deck panels after friction stir welding is complete and while the panels are flat. Each unit will be supported on three longitudinal stringers. The units will each be supported on half of one stringer. In order to accommodate the crowned roadway profile, each unit will need to be deflected during installation to bear uniformly along the length of all three stringers. Upon installation, the panels will bear on two of the stringers with approximately ½" of clearance at the third stringer. A force of approximately 8 kips per purlin is required to deflect the units until they bear uniformly on all three stringers. After each purlin is bolted to the first two stringers using two ¾" diameter ASTM A325 high strength bolts, two

similar bolts at each purlin have sufficient capacity to deflect the panels as the bolts are tightened. The purlin flanges and webs have sufficient strength to resist the forces from the bolt tightening operations. The bolts will be tightened sequentially and in ¼" increments until the purlins are in uniform bearing at each of the stringers. The locked-in stresses due to deformation of the panels are accounted for in the panel design.

- Pairs of short length hot dip galvanized steel angles will be bolted to the stem of each aluminum purlin for connection of the barriers to the deck system. The steel angles will be bolted to the aluminum purlins with a series of mechanically galvanized high-strength bolts in a slip-critical connection. The top of the angles will be flush with the top of the purlin stem and underside of the deck panels. Because of the crowned roadway, the deck and purlins will be sloped at the ends. Tapered bearing plates will be provided between the bottom of the steel traffic barrier base plate and the top of the purlin angles. The angles will be attached to the underside of the deck panels with Lindapter Hollo-bolt blind fasteners. The combined aluminum purlin and steel angles provide 2.80 times the flexural strength, and 1.45 times the stiffness of the existing steel purlin channels.
- Each of the traffic barrier posts are supported directly on and bolted to the purlin steel angles with four high-strength bolts. The aluminum barrier toe clips bear directly on top of the aluminum deck panels. The ¼" x ½" lip on the top of the deck end extrusion below the median barrier must be ground flush in order to accommodate the west side toe clips. Each toe clip is bolted to the purlin steel angles with high-strength bolts by drilling through the aluminum deck. The holes in the deck intersect the top plate, inclined webs, and bottom plate. The toe clips on the east and west side of the median barrier are staggered and bolt to the offset purlins in an alternating pattern.

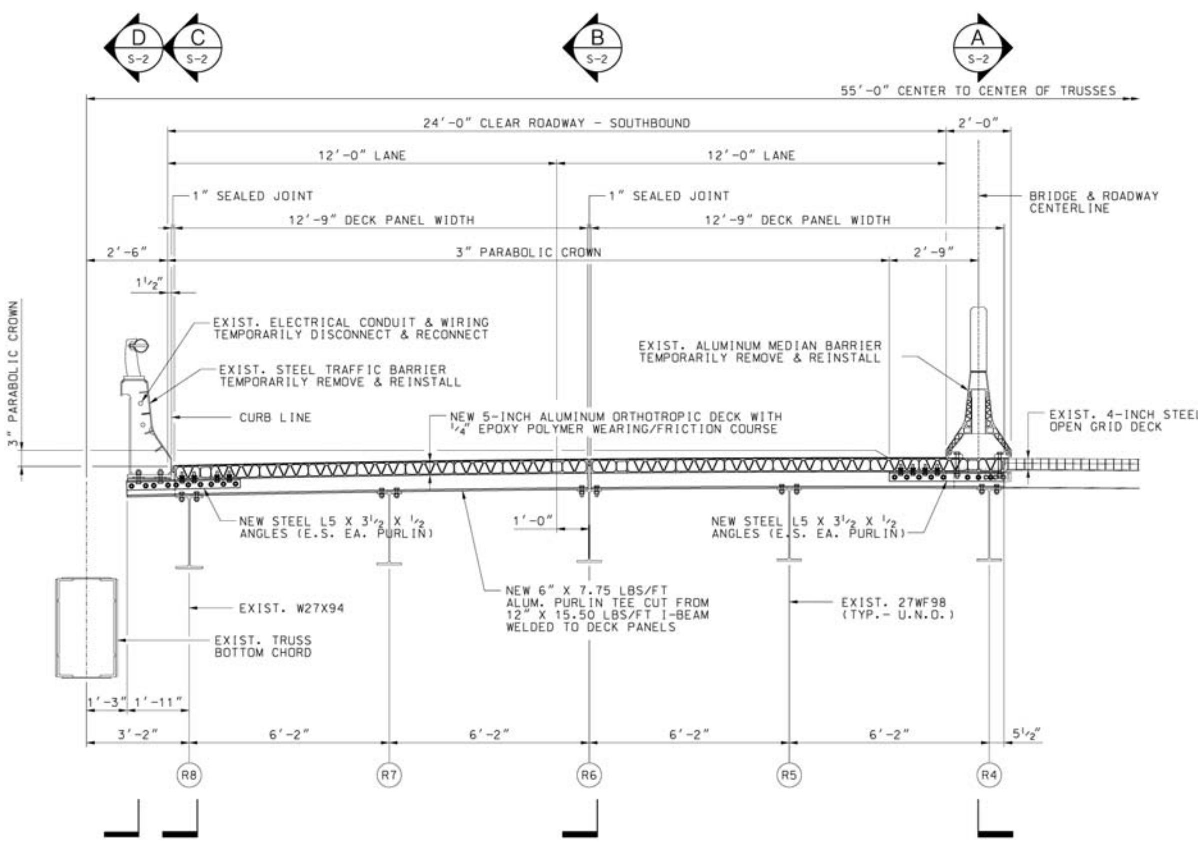

FIGURE 22: Example 2 – Part Transverse Section

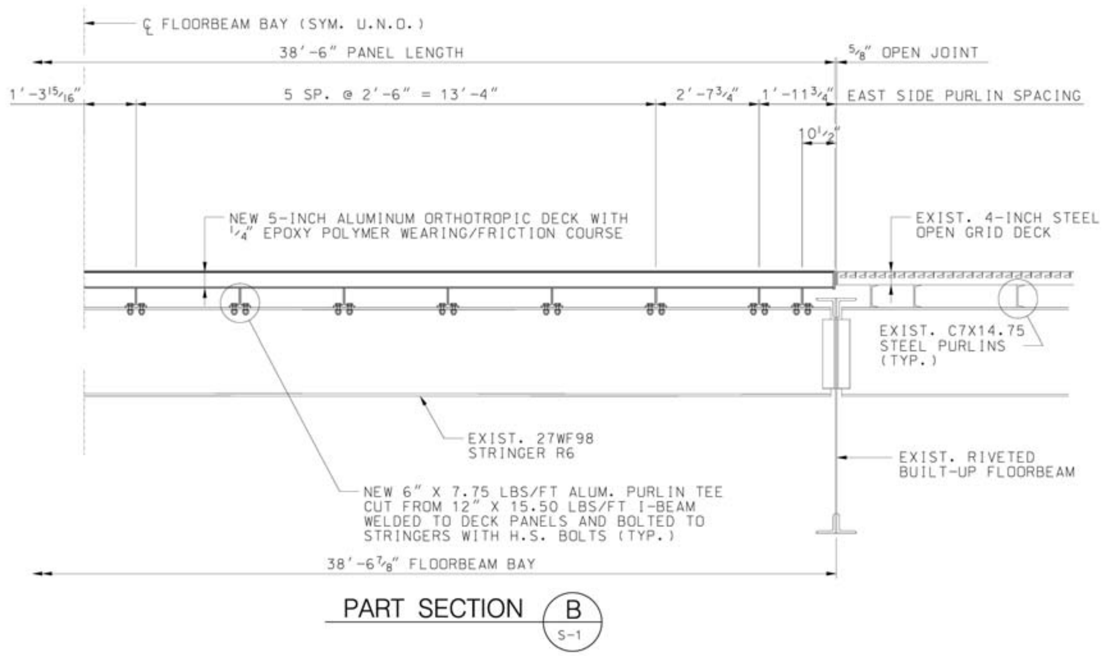

FIGURE 23: Example 2 – Part Longitudinal Section

Future Evolution and Enhancements

Future evolution of the aluminum orthotropic deck system includes use of aluminum stringers, in lieu of steel stringers, welded to the underside of the aluminum orthotropic deck panels as follows:

- Unlike Example 1, above, the deck panels are oriented with the primary direction parallel to the bridge axis (i.e. longitudinal direction). This allows the panels to be configured without intermediate deck panel transverse joints at the stringer mid span.
- Each stringer consists of an inverted tee, with the web and bottom flange fabricated from 6061-T6 aluminum alloy, which has greater yield and tensile strength than the aluminum deck panel 6063-T6 aluminum alloy. The deck panels act as the top flange of the stringers. This configuration locates the weld near the composite neutral axis where stresses are low.
- Stringers will be more closely spaced than the existing stringers to maintain deflection limits with a lower modulus of elasticity and stress limits with lower permissible flexural stresses. A similar or lower weight solution is achieved with the lower density aluminum, which offsets the use of a greater number of stringers. The stringer spacing still permits the stringers to be offset from the existing stringers with similar advantages in ABC. The closer stringer spacing yields lower System 2 Stresses, which addresses the lower permissible flexural stresses in the secondary direction.
- The welded connection of the aluminum stringers to the aluminum deck panels eliminates the numerous fasteners required for the connection of the aluminum deck panels to the steel stringers.
- Aluminum stringers are bolted to the floorbeams using similar details as that described in Example 1, with galvanized steel tees connected to the floorbeam web and connection plates.

Research and Testing

The Florida Department of Transportation in the process of completing a load testing program of two 5-inch aluminum orthotropic deck panels, measuring approximately 8' x 14', connected to three steel stringers using high-strength tension control bolts, and configured similarly to that described in Example

1 above. The test program yielded favorable results. A test report will be available later in 2016. There are plans to install the test set-up on an existing Florida bascule bridge later in 2016.

Acknowledgments

The author would like to express gratitude to Alberto Sardinas of the Florida Department of Transportation – District 4 for his leadership and foresight for establishing the research project responsible for finally solving the steel open grid deck issue and his insight and guidance in developing the aluminum orthotropic deck design.