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Aluminum Orthotropic Deck Update

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Introduction

Owners of bridges with steel open grid deck continue to be plagued with numerous issues including premature fatigue failure of the deck, cracking of attachment welds, traffic and bicycle accidents due to limited skid resistance, premature failure of protective steel coatings and corresponding corrosion of the unprotected steel framing members. A new 5-inch deep, friction stir welded (FSW) aluminum orthotropic deck with a solid riding surface was developed under a Florida Department of Transportation (FDOT) research project, conducted from 2013 to 2015. The primary purpose of this new deck system was to replace 5-inch deep steel open grid deck on a weight-neutral basis and address the safety and long-term durability concerns of the more than 85-year old deck technology. Engineering analysis, laboratory testing and a limited number of installations and prototypes have yielded very promising results. The new deck system is well suited for rapid installation applications, where minimal disruption to roadway and/or navigation traffic is critical.

The new aluminum orthotropic deck system was initially introduced to the movable bridge industry at the 2016 Heavy Movable Structures (HMS) 16th Biennial Symposium in a presentation and corresponding technical paper titled *Aluminum Orthotropic Deck: A Viable Alternative to Steel Open Grid Deck*, by George Patton, PE, MSCE. This paper recaps the various issues with steel open grid deck, ways aluminum orthotropic deck better address these issues, details and results of research performed, and advances and developments of the deck system since the 2016 HMS Symposium.

Aluminum Orthotropic Deck Overview

Steel open grid deck and aluminum orthotropic deck are in a distinct category of ultra-lightweight bridge decks, with unit weights less than 25 psf. Aluminum orthotopic deck is summarized below.

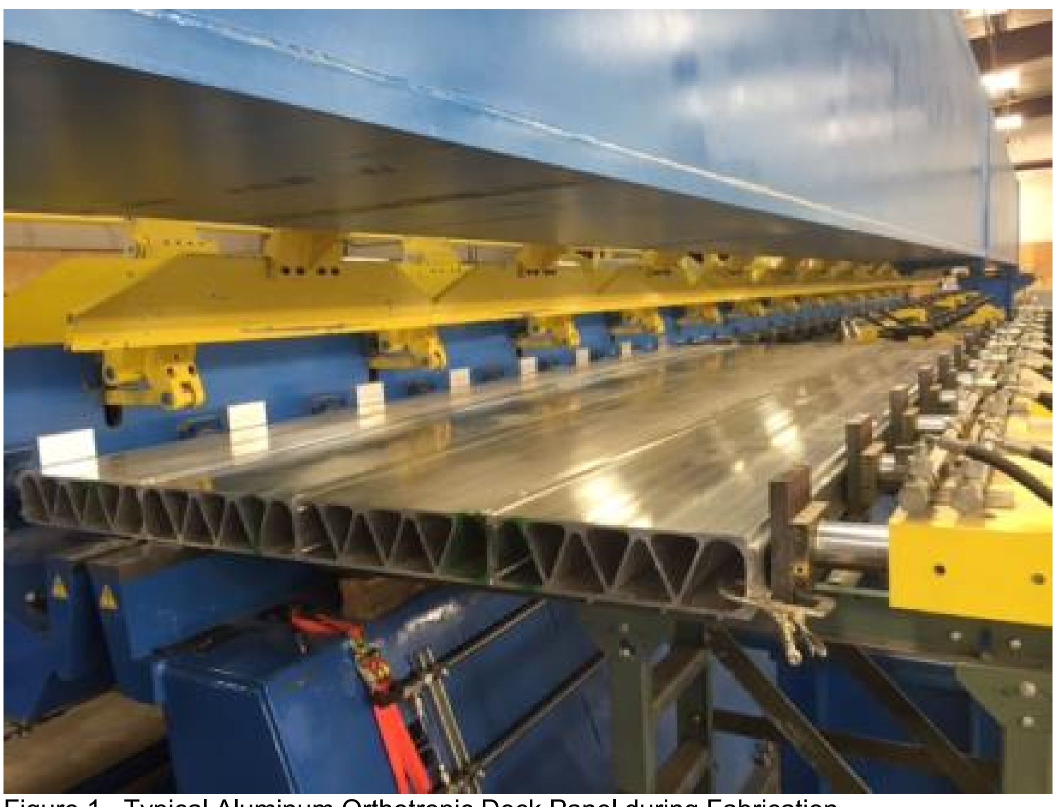

Figure 1. Typical Aluminum Orthotropic Deck Panel during Fabrication

The aluminum orthotropic 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. Intermediate extrusions are typically 18 inches wide with a combination of open (female) and closed (male) ends. End extrusions finish the ends of the deck panels, vary in width to accommodate different panel dimensions, and include a lip at the deck top surface to retain the wearing surface. The ends of the extrusions include a vertical web with seats that act as built-in backing for the FSW complete joint penetration single-sided welds. The female ends are open with cantilevered flanges that mate with the seats of the male ends. Extrusions can be made in lengths up to 40 feet. The individual extrusions are friction stir welded together to create deck panels up to 14 feet wide.

The deck includes 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 with basalt aggregate. The unit weight of the deck without wearing surface and fasteners is approximately 18.3 psf, although weight can vary slightly depending on panel dimensions. With a 1/4-inch thick two-layer thin epoxy polymer friction/wearing course, the unit weight is approximately 21.9 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. A 3/8-inch thick three-layer wearing/friction course, with a longer service life, increases the unit weight to 23.7 psf.

The closed section makes the aluminum orthotropic deck significantly more structurally efficient than steel open grid deck. The deck has a moment of inertia six times that of steel open grid deck, with twice the stiffness, even though the modulus of elasticity of aluminum is roughly one-third that of steel. In addition, the nearly isotropic section properties are such that the moment of inertia in the secondary direction is approximately 90 percent of those in the primary direction. The equivalent strip width (effective width of deck resisting a wheel load) is more than 1.5 times that of steel open grid deck.

Friction-stir welding (FSW) 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 in direct contact with the surface of the plates and a smaller threaded pin that plunges within the depth of the plate at the joint. The firm contact of the tool against the plates and rapid rotation generates heat by friction at the shoulder and pin surfaces. Softening of the material from the heat (between 70 and 90 percent of the melting point of the aluminum) and tool rotation 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 it cools.

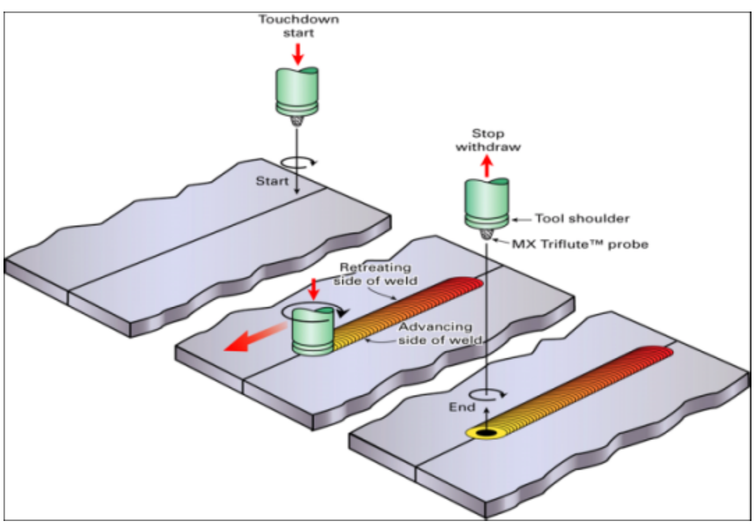

Figure 2. Schematic of Friction Stir Welding Process

Like steel open grid deck, aluminum orthotropic deck is typically supported on and spans transversely across a series of steel stringers, although other configurations are possible and have been used. The maximum recommended stringer spacing is 6.0 feet, typically governed by deflections at the AASHTO LRFD Service I Limit State. Strength and Fatigue Limit States are also considered but do not typically govern. The deck is typically secured to the steel stringers by bolting.

Detailed discussion of the above including structural behavior and methods of structural analysis can be found in the technical paper from the 2016 HMS Symposium.

Steel Open Grid Deck Issues

Steel open grid deck continues to be specified by bridge owners, despite various recurring issues with each installation, and despite the availability of a new deck system that eliminates these concerns. These recurring issues, corresponding causes, and implications are discussed below together with the ways aluminum orthotropic deck better addresses these same issues.

Premature Fatigue Failures

Steel open grid deck commonly experiences premature fatigue failures, especially on bridges subject to significant volumes of heavy truck traffic. These failures are largely due to fatigue prone details (Category E per the AASHTO LRFD Bridge Design Specifications) at the welded intersections of the series of steel grid main bars, tertiary bars, cross bars and diagonal bars. Fatigue cracks at these intersections typically propagate through the cross bars, tertiary bars, and/or diagonal bars resulting in localized loss of these components and corresponding larger holes in the deck. Steel plates are typically welded to the top of the deck to cover these holes. Fatigue cracks have been found relatively early in the service life (less than 20 years) of many bridge decks. There are several factors that contribute to these premature fatigue failures.

Figure 3. Typical Steel Open Grid Deck Fatigue Crack and Welded Repairs

Changes in design and construction practices have led to unconservative estimations of fatigue stress range in steel open grid decks. AASHTO LRFD Bridge Design Specifications and its predecessor AASHTO Standard Specifications for Highway Bridges include a formula to compute the effective strip width of deck that resists an axle or wheel load. (NOTE: In AASHTO Standard Specification, the strip width formula is based on a wheel load in units of tons. In the AASHTO LRFD Specification, the formula is based on an axle load in units of kips, consistent with the use of lane loading throughout the specification. The two specifications yield the same effective strip width.) Per AASHTO LRFD Table 4.6.2.1.3-1, this formula is as follows:

Strip Width (in) = 1.25P + 4.0 S_b, where
P = Axle Load (kips) and
S_b = Spacing of Grid Bars (in)

The formula was developed empirically based on load testing of steel open grid deck panels, with loads placed away from the panel edges. Where the deck is discontinuous, wheel loads cannot distribute across the joint between deck panels. As such, the distribution width for a wheel located next to the panel edge is essentially half that estimated for the load placed away from the panel edge or where the deck is made continuous by way of connections between adjacent panels. Past construction practices included details to connect cross bars of adjacent panels. Alternatively, angles, commonly referred to as chevron bars, were welded between main bars of adjacent panels. At some point, the practice of connecting adjacent panels was discontinued, likely due to the additional cost, duration and challenges to make the numerous welded connections. With a distribution width that is approximately reduced by half, fatigue stress range for a given wheel load is effectively doubled. AASHTO LRFD C6.6.1.2.5 notes that the fatigue resistance, in terms of number of cycles of load, is inversely proportional to the cube-root of the stress range. As such, if the stress range is increased by a factor of 2, the fatigue life decreases by a factor of 2^3 or 8.

Many owners and design engineers may be unaware of the above issue. Currently, there is no commentary in AASHTO LRFD or a bulletin from the Bridge Grid Flooring Manufacturer's Association (BGFMA) that warns of this issue. The BGFMA website includes published tables of deck span limits. However, there is a footnote that states "fatigue is not considered in establishing these values". As fatigue often controls the design of the deck system, these tables should not be used alone to size the deck. Proper design of the deck for fatigue is required to achieve a long service life. Where fatigue is not considered, the service life has shown to be significantly shortened, with deck replacement required at a greater frequency, resulting in corresponding economic impacts and traffic disruptions.

In addition to the above, the stringer layout on many bridges has shown to exacerbate fatigue issues. In many cases, the stringer spacing and locations relative to the travel lanes results in wheel lines straddling the stringers, producing high negative flexure in the deck over the stringers. This yields high live load fatigue stress range at the Category E fatigue detail. Conversely, stringers located along or near the wheel lines produce significantly lower fatigue stress range.

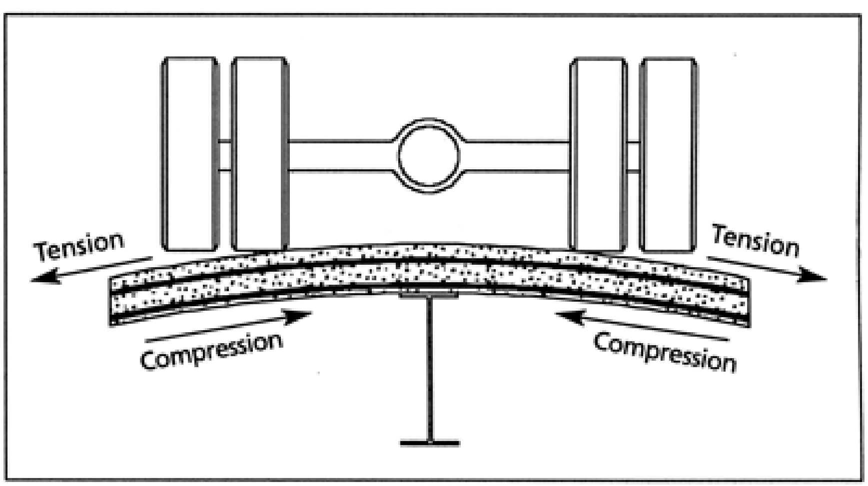

Figure 4. Wheel Line Placement for Maximum Negative Flexure

In recognition of the above fatigue issues with steel open grid deck, aluminum orthotropic deck was designed for infinite fatigue life in accordance with AASHTO LRFD Chapter 7, using worst case boundary and loading conditions. This fatigue resistance considers that deck panels are discrete and discontinuous with no distribution of load across the panel joints and evaluates the fatigue stress range with wheel loads placed at the panel edges, and stringer spacing, wheel line placement, and support conditions configured to produce maximum positive and negative flexure. (NOTE: As aluminum technically does not have a constant amplitude fatigue threshold like steel, infinite fatigue life is considered to equal 5 million cycles of fatigue loading over a 75-year service life.) Detailed discussion on the fatigue evaluation can be found in the technical paper from the 2016 HMS Symposium.

Premature Failure of Attachment Welds

Steel open grid deck commonly experiences cracking of the welds used to attach the deck to supporting steel framing. The weld cracking can be limited to just a few locations or widespread throughout the deck and typically occurs shortly (less than one year) after the new deck is in service. In some cases, welded repairs are effective and stop further cracking, while in other cases, repairs may be ineffective and reoccur or additional cracking may develop. In the latter case, the weld cracking can become a significant maintenance nuisance. Attachment weld cracking is a result of a number of factors.

Attachment welds between the main bars and stringer flanges are typically 1/4-inch fillet welds that are 1-1/2-inch long, which is considered the minimum size for a structural weld. Due to construction tolerances including deck panel longitudinal and transverse camber, rolled steel stringer camber, and stringer erection tolerances, the steel grid deck panels do not bear uniformly on the stringers during installation. Camber or variation from flatness is inherent in steel grid deck panel fabrication. A combination of cross bars, tertiary bars, and diagonal bars offset from the main bar neutral axis, internal restraint within the grid, and shrinkage of welds at the intersections introduces residual stresses that causes panel distortion. Stress relieving from high heat during hot dip galvanizing further contributes to the distortion. Transverse camber can be as much as 0.38 inches for an 8-foot wide panel (0.004 x W x 12) and longitudinal camber can be as much as 0.90 inches for a 25-foot long panel (0.003 x L x 12), which are the maximum recommended dimensions for hot dip galvanized deck. Camber of rolled steel stringers can be as much as 0.10 inches over the 8-foot width of a deck panel (0.125 x W/10). The tops of adjacent stringers can be misaligned by as much as 0.12 inches due to erection tolerances (+/- 0.06). From the above tolerances, camber of steel grid deck panels contributes the most to fit-up issues. To facilitate welding, it is common practice to load the panels in an attempt to achieve uniform bearing while making the welded attachment. Loading of the panels during welding and corresponding unloading of the panels after welding introduces residual stresses in the welds as the panels attempt to spring back to their fabricated geometry. These residual stresses contribute to the attachment weld cracking. Unfortunately, nothing can be done to eliminate the fit-up tolerances and corresponding residual stresses.

Even after loading the panels, gaps between the main bars and stringer flanges are still common. With these root openings, wheel loads are transferred through the relatively small attachment welds rather than through direct bearing between the main bars and stringer flanges. Common attachment details include welding both sides of each main bar, with the welds staggered and located near opposite edges of the stringer flange. With the separation between the staggered welds, each weld is effectively eccentric to the center of the main bar. With this eccentricity, live load transferred through the weld produces torsion that increases live load stresses in the main bars and attachment welds. These stresses are cyclic and contribute to fatigue failure of the welds. Location of the welds on both sides of the main bar at the same location over the stringer web, rather than staggered and at the edge of the flanges, would balance the load and eliminate the torsion and corresponding additional stresses in the weld.

AWS D1.5 - Bridge Welding Code addresses root openings between parts connected with fillet welds. This specification limits root openings to a maximum of 3/16-inch. In cases where root openings exceed 1/16-inch, the leg of the fillet welds must be increased by the amount of the root opening. These provisions have not typically been enforced on steel grid deck attachment welds and thus may be contributing to attachment weld failures. It is recommended that the above root opening provisions be enforced to eliminate this as a factor in the weld failures. Owners should anticipate higher costs for the grid deck installation to account for the additional weld passes to address the above requirements.

Steel open grid deck panels are typically hot dip galvanized and as such the attachment welds should be performed in accordance with the provisions of AWS D19.0 - Welding of Zinc Coated Steel. This specification requires that welds be performed on steel that is free of zinc within a distance of 2 to 4 inches from the weld zone. Zinc coating can be removed by grinding using silicon abrasive discs, burning with a carbon arc or acetylene torch using an oxidizing flame, or shot blasting with portable equipment. However, with the large number of small attachment welds located at the deck underside, removal of the zinc coating is challenging, costly and detrimental to the protective coating system. As a result, it is common practice to weld the deck to the stringers without prior removal of the zinc coatings. This welding has often been performed without qualified weld procedure specifications and qualified welders. As a result, zinc can become entrapped within the heat affected zone, making the weld more brittle and susceptible to weld cracking, especially where welds include residual stress. It is possible to avoid entrapped zinc and achieve a quality weld following specific weld practices. However, these practices increase the cost of deck installation. The key is for the welding to fully volatize or burn off the zinc coating during the welding. Welding should be performed following an approved weld procedure specification, qualified by testing, in accordance with AWS D1.5. The qualification testing should include the thickest anticipated zinc coating, maximum permitted root opening in the joint, low-silicon electrodes (0.2Si or lower), slower welding speeds, and higher heat input. Many welders recommend E6010 or E6011 electrode, which creates a 60 to 70 percent stronger arc that is more effective in volatizing the zinc coating. Weld electrodes should be applied slower than normal in a "whip and pause" action. With this approach, the weld electrode is moved forward 1 to 3 rod diameters along the weld seam and over the zinc coating to volatize the zinc before the weld pool arrives. The electrode is then brought back and paused to fill the puddle. This action is repeated along the weld. As this welding requires finesse and a trained hand, the welder should be qualified in accordance with AWS D1.5 specifically for the approved weld procedure. To confirm weld quality, all welds should be visually inspected and a minimum of 10 percent of the welds should receive non-destructive testing including magnetic particle (MT) and/or penetrant testing (PT). These requirements should be included in the contract documents and enforced during construction. Owners should anticipate higher costs for the grid deck installation to account for the higher required welding expertise and to cover costs for weld procedure and welder qualifications, weld inspection and non-destructive testing, and greater number of identified defects that must be corrected.

Aluminum orthotropic deck is bolted to the steel framing rather than welded and thus eliminates the above weld cracking issues. The configuration of the aluminum extrusions and welded joints between extrusions, and use of friction stir welding, inherently creates nearly flat panels (within 0.12 inches) that minimizes the amount of distortion required to make the panels bear uniformly on the supports. Distortion is mostly limited to stringer camber, which is generally a maximum of 0.18 inches over the 14-foot panel width (0.125 x W/10) and a maximum of 0.12 inches of misalignment between adjacent stringers (+/- 0.06). The nearly flat panels are achieved as a result of several factors. Current extrusions are effectively axisymmetric with the neutral axis at mid-depth, welds balanced about the neutral axis, with top and bottom welds performed simultaneously. Furthermore, friction stir welding introduces much less heat into the joint with corresponding lower weld shrinkage and residual stress. The limited number of bolts used to attach the panels can easily distort the panels the relatively small amount required to achieve uniform bearing with a very low (less than 3 kips) residual force in each bolt.

Limited Skid Resistance

Despite the serrated top surface of steel open grid deck, which initially yields reasonable friction values, these surfaces are known to quickly polish under contact with rubber tires, resulting in a significant drop in friction over a short period (less than one year) after the new deck is placed into service. The already limited skid resistance of steel open grid decks is further reduced in wet conditions, prompting owners to implement warning signs at these bridges. The slippery conditions are even more of a concern for less stable, two wheeled motorcycles and bicycles. Numerous attempts have been made to mitigate loss in skid resistance, including addition of small, welded studs and roughening the surface with a scabbler. These solutions have yielded minimal improvements with rapid loss in skid resistance and have damaged tires. The poor skid resistance is responsible for contributing to vehicle and bicycle accidents with corresponding economic impacts to the community.

Aluminum orthotropic deck includes a skid-resistant wearing/friction course applied to the top of the panels. FDOT performed skid-resistance testing of the wearing/friction course that yielded positive friction results. (See Heavy Vehicle Simulator Testing section below.) The properly applied wearing/friction course is durable with a relatively long service life of 10 to 15 years for a two-layer system and 15 to 20 years for a three-layer system. As it wears, it can be periodically refreshed by cleaning the surface with a simple waterblast and applying another layer on top of the existing surface. The improved skid resistance reduces accidents and corresponding traffic impacts and economic impacts to the community from personal injury and property damage.

Premature Steel Coating Failures and Corrosion

Steel open grid deck provides no protection of the supporting steel members from the environment and thus contributes to the premature degradation and shortened service life of the steel coating systems. Where the steel coating systems are not maintained, the steel open grid deck contributes to accelerated corrosion of the steel framing members. More frequent reapplication of the steel coating system and greater amount of steel repairs increases maintenance costs, increases traffic disruptions, and reduces the bridge service life.

The solid surface of the aluminum orthotropic deck protects the steel coating system and steel framing members from corrosion far better than the steel open grid deck. Although the aluminum deck system includes joints between panels, these narrow joints can be sealed with durable and economical low modulus silicone sealant and polyethylene backer rod. The narrow (less than 1/2-inch) wide joints and minimal thermal movement at these joints maximizes durability and service life of the joints by minimizing stress in the sealant.

The aluminum orthotropic deck system includes details to prevent galvanic corrosion by insulating contact between the aluminum deck, steel stringers and steel mounting hardware. Insulation includes the steel zinc-rich coatings, elastomeric pads or heavy duty nylon tape between the materials at the faying surfaces, and nylon sleeves around bolt shanks. Even with incidental contact resulting from degradation of the insulating materials or coatings, the nature of galvanic corrosion is such that the less noble (anodic) aluminum deck sacrifices itself to protect the more noble (cathodic) steel stringers and hardware. The significantly greater surface area of the deck compared to the steel stringers and hardware yields a very low current density and slow corrosion rate. The 6063-T6 aluminum alloy is inherently corrosion resistant as a result of a tightly bonded aluminum oxide film that forms on the surface. This film 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 small fraction of the material thickness. Pitting is primarily an aesthetic problem that generally does not affect the strength of the aluminum. Detailed discussion on the galvanic corrosion can be found in the technical paper from the 2016 HMS Symposium.

The improved corrosion resistance of the aluminum orthotropic deck reduces maintenance costs and corresponding traffic impacts and increases the service life of the bridge.

Laboratory Tests

Following its development through the research project, FDOT continued with a comprehensive evaluation of the aluminum deck prototype panel to validate the analytical work behind its development, investigate one method of rapid installation, and assess in-service performance. The testing was a two-phased program. Phase 1 consisted of a series of laboratory tests performed at the FDOT Structures Research Center in Tallahassee, FL (fixed-position load testing), and the FDOT State Materials Office in Gainesville, FL (heavy vehicle simulator and accelerated corrosion testing). Phase 2 consisted of field installation and evaluation of the prototype panel under active traffic on the existing North Bridge, a two-lane, double-leaf bascule bridge, in Fort Pierce, FL.

The prototype panel, illustrated below, included two aluminum deck panels, one fabricated using Gen-I extrusions and one fabricated using Gen-II extrusions. The prototype panel was subjected to a series of tests to verify performance, validate response relative to analytical results, demonstrate constructability of the deck-stringer system, and confirm that the deck system was suitable for field testing under active traffic. The two aluminum deck panels were bolted to three rolled steel stringers with a slip-resistant connection and tested simultaneously as a deck-stringer system to emulate a real-world application. The load test program included fixed-position static loading to validate whether actual stresses and deflections were consistent with those from the earlier finite element analysis and within AASHTO LRFD permissible limits. Fixed-position dynamic loading was used to evaluate fatigue resistance. The aluminum deck panels were instrumented with a series of strain gauges to measure the stress under various maximum anticipated loading conditions. Panel deflection was also instrumented and measured. The

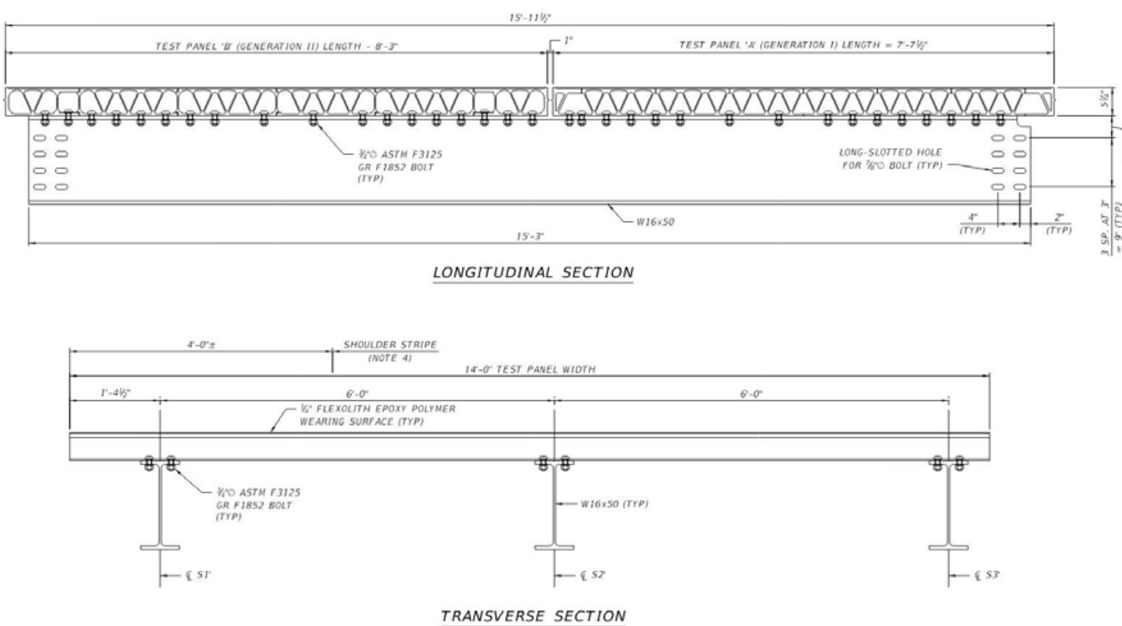

Figure 5. FDOT Aluminum Orthotropic Deck Prototype Panel Assembly

prototype was then subject to FDOT's heavy vehicle simulator to demonstrate performance of the system under heavy moving wheels. (NOTE: Heavy moving wheel loading approximates actual in service conditions and can identify issues that fixed-position loading cannot. A number of ultra-lightweight deck systems performed successfully under fixed-position static and dynamic load testing, and then later failed under heavy moving wheel testing or actual in service conditions. As such, heavy moving wheel testing is critical to evaluation of this class of deck systems.) Additional testing was performed on the wearing/friction course including pull-off tests to evaluate adhesion, friction or skid resistance, and wear assessments. Lastly, accelerated galvanic corrosion testing was performed on fasteners used to connect the aluminum deck to the steel stringers. This section briefly summarizes the series of tests and results. A detailed discussion of the laboratory testing can be found in *Aluminum Lightweight Orthotropic Deck Evaluation Project* by Christina Freeman, PE, and William Potter, PE, published by FDOT in 2017.

The prototype panel assembly was instrumented with 92 strain gauges and 20 deflection gauges. The assembly was tested on both rigid (strong floor) and flexible (elastomeric bearing pads) supports to envelope the effects of variable support conditions. Applied wheel loads included multiple presence and impact factors. Magnitudes were in accordance with the HL-93 design truck and tandem loads at the Service I and Strength I load combinations, the FDOT FL-120 permit truck at the Strength II load combination, and HL-93 fatigue truck at the Fatigue I load combination. Loads were applied in both single-point (wheel) and multiple-point (one or two axle) configurations, with one wheel or axle located adjacent to the edge of the panel, positioned to maximize positive and negative flexure.

Static Load Test

For the fixed-position static load tests, loads were slowly applied and removed, during which deflections and strains were measured with the strain and deflection gauges. The maximum demand-capacity ratio was 0.70 for tension and 0.57 for compression, obtained from the strain gauge measurements, for loading at AASHTO LRFD Strength load combinations. This demonstrated that the deck design is very conservative and has significant reserve strength. The maximum measured deflection at the edge of the panel, mid-distance between stringers spaced 6 feet on center, for loading at the AASHTO LRFD Service II load combination, was 0.14 inches. When accounting for the difference in load factors between Service I and Service II load combinations (1.00 vs. 1.30, respectively), the Service I deflection is 0.11 inches, which is slightly above the AASHTO LRFD prescribed limit of 0.09 inches (L/800). Per AASHTO LRFD C2.5.2.6.1, "Service load deformations may cause deterioration of wearing surfaces in metal bridges that could impair serviceability and durability, even if self-limiting and not a potential source of collapse." Per AASHTO LRFD C9.5.2, "The primary objective of curtailing excessive deck deformation is to prevent breakup and loss of the wearing surface. No overall limit can be specified because such limit is a function of the composition of the wearing surface and the adhesion between the deck and the wearing surface. The limits should be established by testing." As adhesion testing of the wearing/friction course was successful after 600,000 heavy wheel passes under severe conditions, the deflection is not a concern.

Dynamic Load Test

The fixed-position position dynamic fatigue truck loading was applied continuously at 45 cycles per minute for 2 million cycles during a month-long test. The largest measured range of strain occurred at a single gauge located at a weld seam in the top of the deck and was effectively equal to (less than 1 percent over) the constant-amplitude fatigue threshold for a Category E fatigue detail per AASHTO LRFD 7.6.1 for a welded joint with stresses normal to weld axis. The deflections and corresponding panel stiffness remained essentially constant during the duration of the test, which is an indication that the panels did not experience fatigue damage. The number of cycles of fatigue loading was purposely less than the equivalent number of cycles for infinite fatigue life, so that there was reserve fatigue resistance for the field testing.

Heavy Vehicle Simulation Test

The Heavy Vehicle Simulator (HVS), typically used by FDOT to evaluate asphalt pavement, consists of a single loaded tire that runs back and forth across a short stretch of pavement to evaluate performance. For this test, the wheel was loaded to 11.0 kips, and applied to the prototype panel for a total of 600,000 cycles. The magnitude of the HVS load is approximately one-fifth of the loads applied during the strength testing. Therefore, the primary focus of this test was to assess the performance of the wearing/friction course under the effects of repetitive traffic loading (adhesion, wear, and friction), with a secondary objective of observing additional structural response to a moving load that applies pressure to the deck panels in a manner not explicitly addressed in the fixed-position static and dynamic load tests. Effects of heat and moisture were considered (though not at the same time) by heating the panel to between 100°F and 120°F during the first half of the test (300,000 cycles) and saturating the wheel path with water during the second half (300,000 cycles.)

In service structural and fatigue responses from the strength tests were effectively validated by the HVS test, with no observed issues. One observation of note during the HVS test was minor measured slip at the panel-stringer interface while the wheel was moving. However, as the magnitude of the measured load-induced slip was 100 times smaller than anticipated slip from restrained differential thermal movement, the slip was deemed to be inconsequential to deck system performance.

Prior to transporting the panel assembly to Gainesville for HVS testing, wearing surface pull-off adhesion tests were performed to establish a baseline for reference and comparison with results following the moving wheel test. Despite evidence of wear within the path of the loaded wheel and a moderate reduction in bond strength when excessive heat is applied, adhesion test results before and after the moving load test suggest the bond between the overlay and the aluminum panel will perform well under service conditions.

Baseline friction testing was performed prior to the HVS test. Measurements were also taken midway through the test and at the end to assess degradation. Dynamic Friction Testing was performed in accordance with ASTM E1911, using the HVS, as noted above. The measured coefficient of friction values ranged from initial values of 0.85 to 0.90 (Equivalent FN40R Friction Number of 95) to a range of 0.71 to 0.77 (Equivalent FN40R Friction Number of 79) after 300,000 passes in dry conditions, and a range of 0.65 to 0.72 (Equivalent FN40R Friction Number of 72) after another 300,000 passes in wet conditions. These values exceed the required Equivalent FN40R Friction Number of 55. FDOT also

Figure 6. Load Test Setup (photo: FDOT)

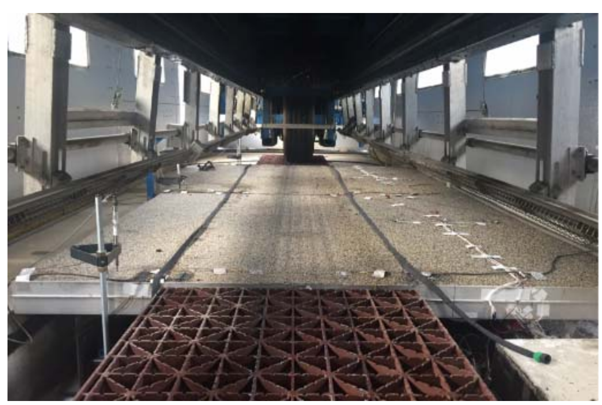

Figure 7. HVS Setup (photo: FDOT)

performed Circular Track Meter Testing in accordance with ASTM E2157 and Texture Meter Testing in accordance with ISO 13473. Both of these tests yield macrotexture mean profile depth. These tests yielded average mean profile depths significantly greater than newer in-service longitudinally diamond ground concrete pavement values (0.6 mm) and longitudinally ground and transversely grooved concrete bridge deck values (1.3 mm). Measured values from the Circular Track Meter were 2.02 mm initially, 1.84 mm after 300,000 passes in dry conditions, and 1.84 mm after 300,000 passes in wet conditions. Measured values from the Texture Meter were 1.82 mm initially, 1.72 mm after 300,000 passes in dry conditions, and 1.66 mm after 300,000 passes in wet conditions. The coefficient of friction and mean profile depth dropped only modestly (20 percent and 9 percent, respectively) by the end of the test while continuing to exceed values typically achieved with new concrete pavement and bridge deck construction.

Corrosion Test

Accelerated corrosion testing was performed on one type of fastener used to attach the deck to the stringers. The testing was completed following publication of the report by Freeman and Potter. Results from the State Materials Office were provided by Christina Freeman for this paper. Corrosion testing concluded in May 2017 after 11 months of continuous immersion in saltwater using Lindapter Hollo-Bolts in both undeformed (pre-installed) and deformed (installed) conditions. Except for some minor oxidation of the aluminum, exhibited by discoloration, and formation of zinc oxide from the zinc plating, no significant long-term corrosion concerns were identified. Based on these results, inspection of these connections at the same frequency as the routine biennial inspections is sufficient.

In-service corrosion is anticipated to be less severe than that from the accelerated corrosion testing, as in-service environmental conditions, consisting primarily of light salt spray or mist, will not be as severe as the full, long-term immersion in salt water, used in the testing. In addition, the actual aluminum deck area is significantly greater than the small aluminum samples used in the testing. Galvanic corrosion current densities are relatively low when the area of the anode (less noble component) is much larger than the area of the cathode (more noble component). (NOTE: The Lindapter Hollo-bolts used in the accelerated corrosion testing are not the primary fastener used to connect the aluminum deck to the stringers. Tension control (TC) bolts are used for that connection.) As the Lindapter Hollo-bolts are zinc plated, with a much lower zinc content than mechanically galvanized TC bolts, and because testing did not electrically isolate the fasteners like the current fastener details, the accelerated corrosion testing is considered very conservative. Recommended fastener details are anticipated to experience less severe effects from corrosion.

Field Test

North Bridge in Fort Pierce, FL is a double-leaf bascule bridge with steel open grid deck. When the research project began, it was scheduled for replacement and was identified as a proving ground to test deck panel installation and evaluate in-service performance for the remaining life of the bridge. The test panel assembly was detailed and fabricated specifically for installation on that bridge. Using the deck panel and installation concepts developed the by the H&H research team, HDR worked with FDOT District 4 to develop an installation and monitoring plan.

Installation

A key factor for any system to function as a viable deck replacement alternative is the ability to install it with minimal disruptions to roadway and marine traffic. Having validated a viable panel assembly in the laboratory, the field test was aimed at vetting rapid installation using accelerated bridge construction (ABC) concepts.

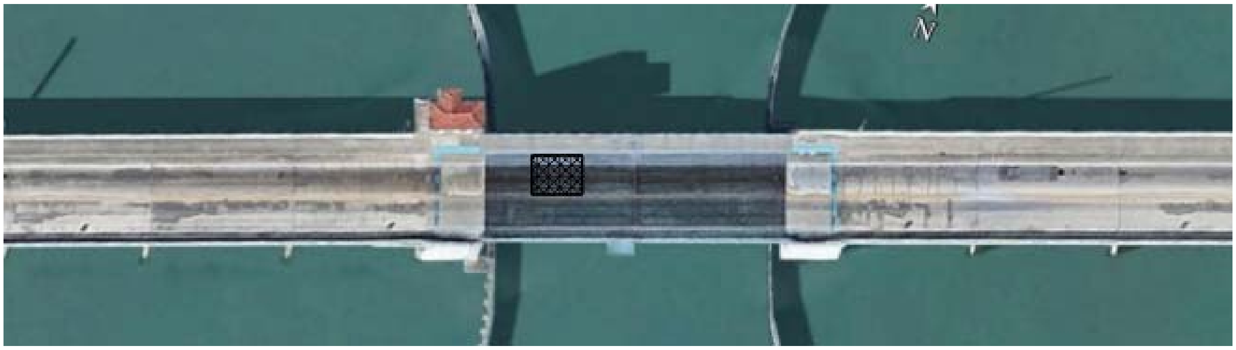

Figure 8. Prototype Panel Field Installation Location Plan on North Bridge

Deterioration patterns in the concrete and steel deck over the full length of the bridge and bascule span steel framing are consistent with traffic exiting the boat ramp to the east and leaving a trail of saltwater across the bridge, which creates a more aggressive environmental condition for the deck in the westbound lane. Therefore, the section of deck targeted for this evaluation was in the westbound lane on the west leaf, between Floorbeams 2 and 3. Even though a single aluminum deck panel could have easily spanned the full roadway width, the prototype panel was intentionally sized to cover half of the roadway (curb line to centerline) over one floorbeam bay to permit one lane of traffic during installation.

The general concept was to remove the target section of existing deck (steel open grid deck welded to a series of stringers) and replace it with the prototype panel assembly (aluminum deck panels bolted to a series of stringers). The replacement was to occur during an overnight work shift with intermittent full roadway closures and a continuous navigation channel closure from 9 pm to 7 am.

Given the experimental nature of this installation, the plan was to include provisions for returning the bridge to its original condition at any time during or after construction in the event installation could not be completed as planned or the installed panel failed to perform satisfactorily. The construction plans were developed to maximize the amount of work that could be performed prior to the closures so work within the closure would be limited to replacement of the deck and stringer as whole assemblies. Prior to removal, the following work took place:

- Connection brackets, spaced to align with the prototype panel stringers, were installed on the floorbeams offset from the existing stringers
- Existing stringer end connections were modified to permit vertical movement during removal and (contingent) reinstallation.
- Centerline stringer supplemental flange angle was installed to permit removal of half the stringer flange.

The existing stringer end connection modifications essentially replicate the existing floorbeam web face-mounted double-angle connection on a parallel surface offset from the face of the web, beyond the edge

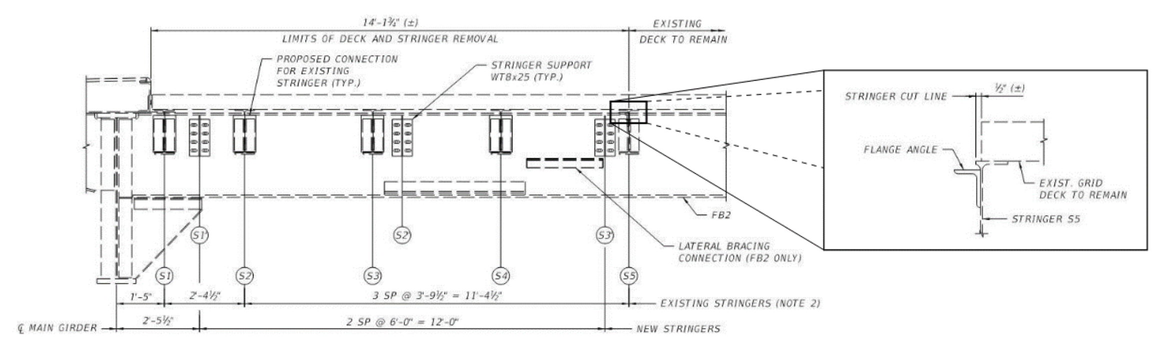

Figure 9. Pre-closure Installation Details

of the flange. The ends of the stringer web were cut within the cope beyond the edge of the floorbeam flange. The portion of stringer web between the existing clip angles were removed. The stems of new WTs were bolted between the clip angles and the WT flanges served as the new mounting face for new clip angles bolted to stringer webs. These modifications made it possible for the stringer to be unbolted and lifted vertically without interference with the floorbeam flanges during removal and reinstallation, if necessary. Once it is confirmed that deck performance is acceptable, the WTs and clip angles can eventually be removed to minimize residual weight from the existing non-functional elements.

Pre-closure work to prepare the existing deck for removal began in late February 2021 and lasted approximately two weeks under single-leaf operation for navigation and with no disruptions to roadway traffic. Traffic on the bridge was reduced to one lane on the evening of March 11 to make final preparations and ultimately remove and replace the deck. By 6 am on the morning of March 12, the existing deck panel was extracted, and the prototype panel was in place two hours later. Final stringer connections were completed, and the bridge was re-opened to two-way traffic by mid-morning, a few hours behind schedule.

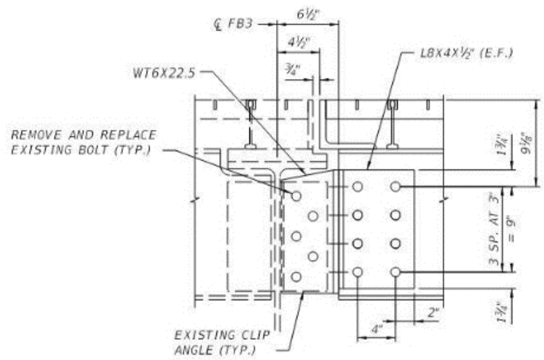

Figure 10. End Connection Modifications

Overall, the installation was successful. It demonstrated that a well-executed, detailed plan for removal and replacement of an existing section of steel open grid deck on stringers with a new section of aluminum deck on stringers is both feasible and amenable to rapid-deployment to minimize user disruptions and costs. Limited delays on this installation were primarily attributed to the experimental nature of the exercise and "surgical" removal work required for partial deck replacement, factors that would not otherwise be applicable to a full deck replacement. Finishing the work with a high volume of traffic in the adjacent lane proved inefficient, underscoring the importance of completing the work during the overnight closure as planned.

The contract also included requirements for cleaning and preparing the existing panel for future redeployment, if necessary, including welded bearing angles, miscellaneous weld repair, and spot coating repair.

Routine Monitoring

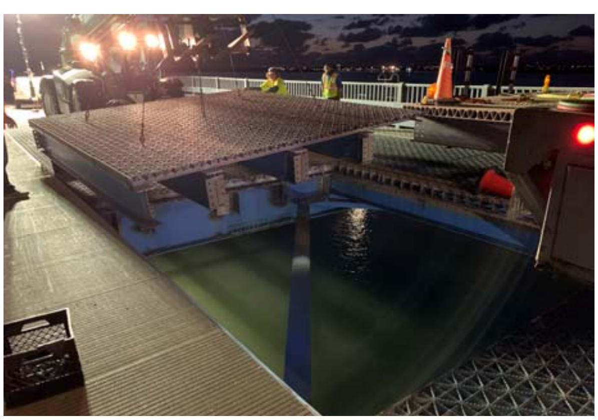

Figure 11. Existing Deck and Stringer Removal

Figure 12. Prototype Panel Installed

Following installation, HDR began performing regular inspections, initially on a three-month interval that expanded to six months after subsequent site visits produced no discernible evidence of changes in the condition or performance of the prototype panel. The deck panel system continues to perform in accordance with expectations based on the analytical development efforts and laboratory testing.

Latest Updates on Aluminum Orthotropic Deck

New Gen-III Extrusions

Following panel fabrication for two recent projects (Browns Park NWR Suspension Bridge, NW Colorado and Marine Parkway Bridge Prototype Panels, Queens, NY) the aluminum extruder and company that performed the friction stir welding offered valuable feedback that led to suggested changes and development of the Gen-III extrusions. These modifications are anticipated to further simplify fabrication and reduce cost.

Feedback noted that the asymmetry and relatively long cantilevered flanges at the open (female) ends of the Gen-II intermediate and end extrusions resulted in minor distortion of the flanges during extruding. This distortion required roll correction for proper fit-up during friction stir welding into panels that added time and labor cost to the fabrication process. Suggested changes include addition of a vertical web at the female ends that shorten and stiffen the cantilevered flanges. The vertical web with built-in seats for friction stir welding replace the open end of the Gen-II end extrusion. To adapt the rest of the panel for this change, a female-female extrusion (open at both ends) is required. Feedback also included recommendations to eliminate trimming of the flanges of the end extrusions to accommodate an infinite range in panel width dimensions. Aluminum affords the ability to extrude any number of complex shapes, readily and economically. New end extrusion geometry includes a void between two vertical webs that can be adjusted in width from 1.75 to 4.0 inches (a variation in width of 2.25 inches) with a corresponding full range in end extrusion widths from 11.25 to 13.5 inches. Together with intermediate extrusions with widths of 18 and 13.5 inches, deck panels can be configured in an infinite range in deck panel widths. A new die is required for each extrusion.

As a new set of dies was required for the Gen-III extrusions, an additional adjustment was made to the sections to further improve fatigue resistance. The series of voids in each extrusion is lowered by 1/16-inch such that the thinnest portion of the top plate is increased 25 percent from 1/4-inch to 5/16-inch. In conjunction, the bottom plate thickness is reduced from 3/8-inch to 5/16-inch. This minor change nearly doubles the top plate section modulus and corresponding flexural resistance, without increasing the deck weight. The top plate, which is subject to wheel patch loading, benefits from this change by reducing the top plate flexural stresses by 56 percent. As the bottom plate is not subject to wheel patch loading, and does not govern the design, the 44 percent decrease in bottom plate thickness is inconsequential.

TABLE 1 – GEN-III BASIC EXTRUSION PRIMARY DIRECTION SECTION PROPERTIES

Parameter	18-inch Male-Female Intermediate Extrusion	18-inch Female-Female Intermediate Extrusion	13.5-inch End Extrusion
Area, A	23.34 in^2	21.47 in^2	19.17 in^2
Moment of Inertia, I_x	93.52 in^4 (62.34 in^4/ft)	89.73 in^4 (59.82 in^4/ft)	72.70 in^4 (64.62 in^4/ft)
Neutral Axis Ref., y_{top}	2.406 in	2.397 in	2.443 in
Neutral Axis Ref., y_{bott}	2.594 in	2.603 in	2.557 in
Section Modulus, S_{xtop}	39.87 in^3 (25.91 in^3/ft)	37.43 in^3 (24.96 in^3/ft)	29.76 in^3 (26.45 in^3/ft)
Section Modulus, S_{xbott}	36.05 in^3 (24.04 in^3/ft)	34.47 in^3 (22.98 in^3/ft)	28.43 in^3 (25.27 in^3/ft)
Weight (w/o Wear. Course)	27.3 plf (18.2 psf)	25.1 plf (16.8 psf)	22.4 plf (19.9 psf)
Weight (w/ Wear. Course)	32.6 plf (21.7 psf)	30.4 plf (20.3 psf)	26.3 plf (23.4 psf)

NOTE: A typical 12.75-foot wide panel with 6 ~ Male-Female Extrusions, 1 ~ Female-Female Extrusion and 2 ~ End Extrusions has a unit weight of 18.3 psf w/o Wear. Course and 21.8 psf w/ Wear. Course.

The modifications from Gen-II to Gen-III extrusions slightly increase deck weight and section properties. For the 18-inch male-female extrusion, which represents the majority of extrusions in deck panels, moment of inertia increased approximately 7 percent from 58.09 in^4/ft to 62.34 in^4/ft, bottom section modulus increased approximately 2 percent from 23.49 in^3/ft to 24.04 in^3/ft, and top section modulus increased approximately 13 percent from 22.99 in^3/ft to 25.91in^3/ft. Unit weight for this extrusion increased approximately 0.8 psf (5 percent) from 17.4 psf to 18.2 psf.

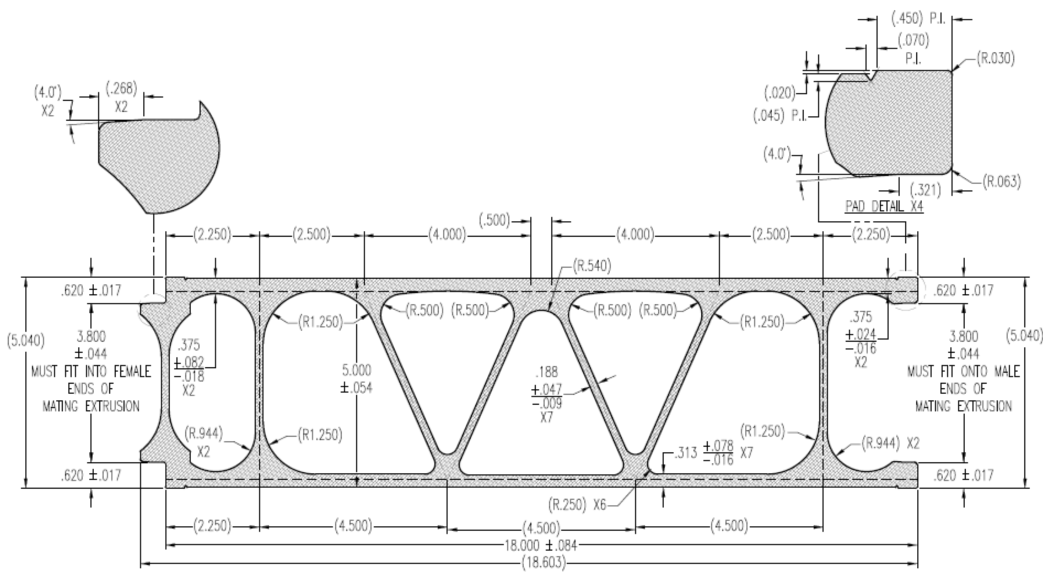

Figure 13. Gen-III 18-inch Male-Female Intermediate Extrusion

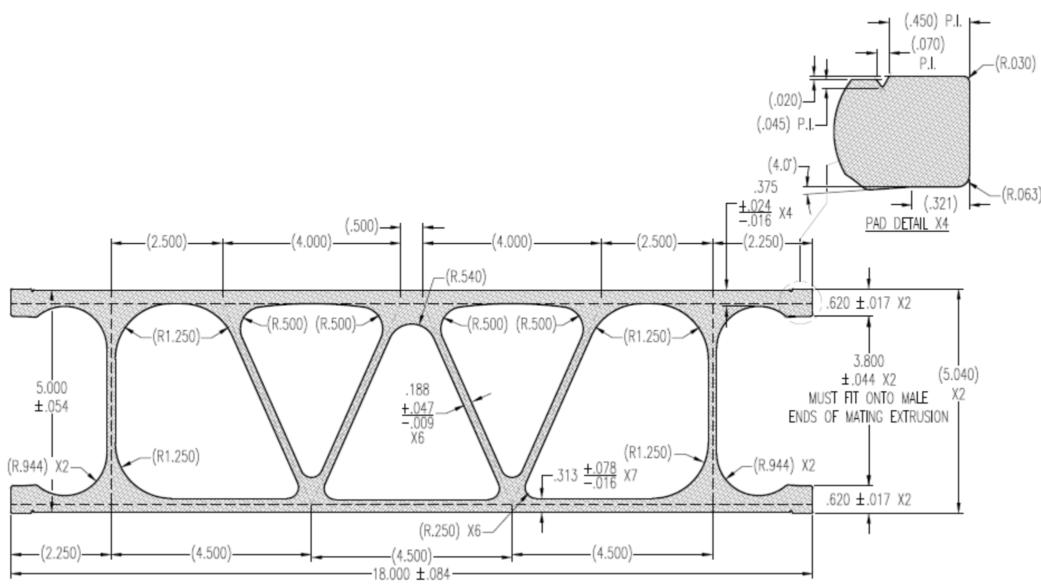

Figure 14. Gen-III 18-inch Female-Female Intermediate Extrusion

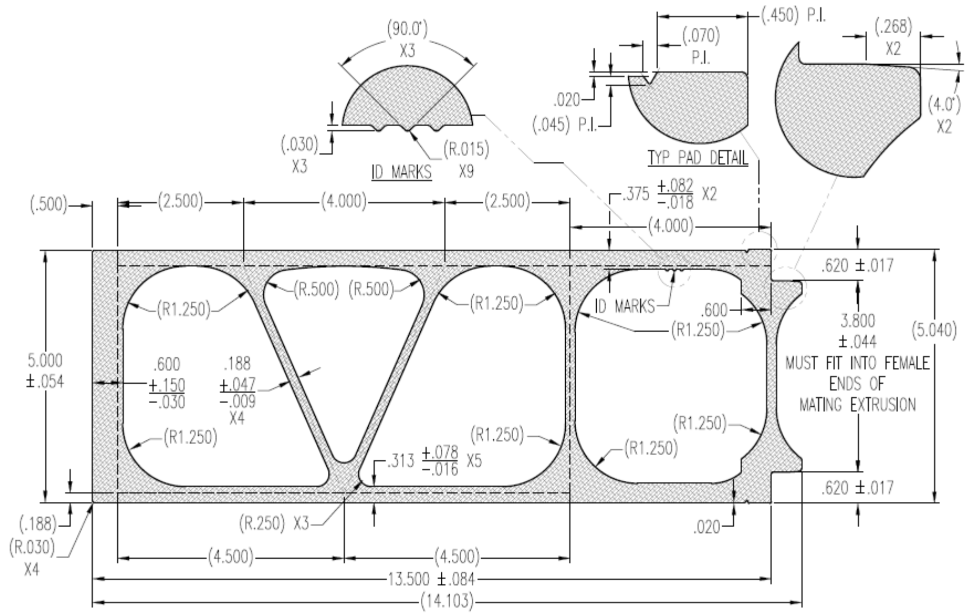

Figure 15. Gen-III 13.5-inch End Extrusion

Updated Connection Methodology and Details

In the original FDOT research, a decision was made to use a slip resistant connection between the aluminum orthotropic deck and steel stringers. This decision was based on the concern that slip between the deck and stringer would result in fretting of the steel protective coatings and eventually result in a corrosion issue. As these connections were further developed, it was discovered that the connection design was overly complicated and yielded a large number of fasteners. There are several factors that complicated the connection design including discrete deck panels with transverse joints at intermediate points along the stringer that give rise to spikes in horizontal shear, relatively large horizontal shear forces from a combination of live load and restrained differential thermal movement, and restrictive geometry including narrow stringer flange widths and series of internal webs within the deck that restrict the number and pitch of fasteners. The slip resistant connection requires use of high strength bolts tensioned to a specified minimum proof load. Although use of tension control (TC) bolts addressed the inability to hold the head of the bolt within the hollow section, with bolts tensioned from the deck underside, bolt installation introduced other challenges. Delivery of the bolts down the length of the hollow extrusions to the location of the holes was cumbersome and required special tools. Blind fasteners, such as Lindapter Hollo-bolts, huck bolts, and Rivnuts, designed to work with hollow sections, did not provide the specified proof load required to achieve the slip resistant connection. Lastly, the deck surface at the stringer locations required shot blasting to achieve the required coefficient of friction for the Class B slip surface, which added cost and time.

With the above challenges, the original decision to use a slip resistant connection was discarded, and a simpler and more cost effective clamp-type connection was developed that secures the deck, while permitting the connection to slip, which relieves forces from restrained differential thermal movement, and without detrimental effects from millions of cycles of live load movement. This new approach and details for attaching the deck to the steel stringers is described below.

Bolted clamps can consist of either commercially available hardware such as galvanized Lindapter LS clamps or custom fabricated galvanized steel or aluminum clamps. The Lindapter LS clamps are detailed to accommodate a range of different grip lengths. Custom fabricated clamps can be provided with shims for adjustment to achieve uniform bearing and clamping action. As the clamps are used only to secure the deck, fewer fasteners are required than for a slip resistant connection. Clamps are recommended on each side of the stringer flange at a minimum of three locations (each edge and the center of a deck panel). A greater number and closer spacing may be considered for additional clamping action. Both types of clamps have been successfully used in practice with the custom fabricated clamps used on the Browns Park NWR Suspension Bridge over the Green River in NW Colorado and the Lindapter LS clamps used in the Bridgestone Americas Tire Operations Factory in Aiken South Carolina.

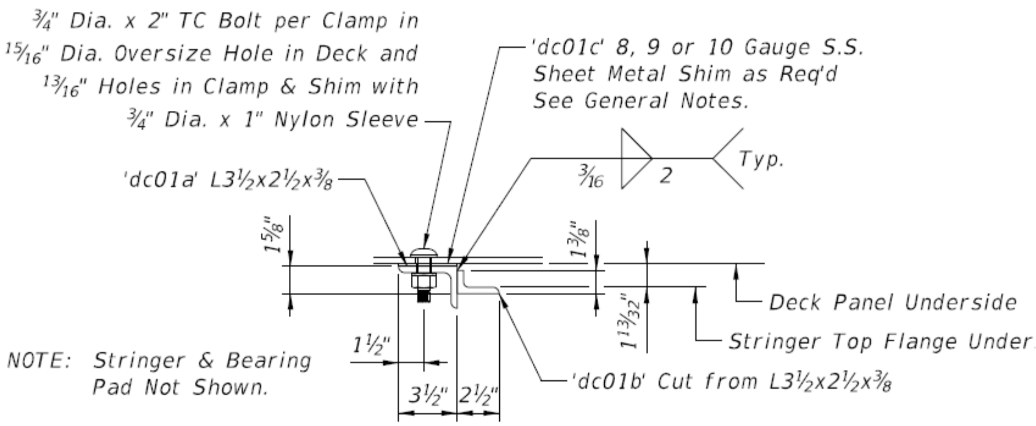

Figure 16. Custom Clamp Detail (Browns Park NWR Suspension Bridge)

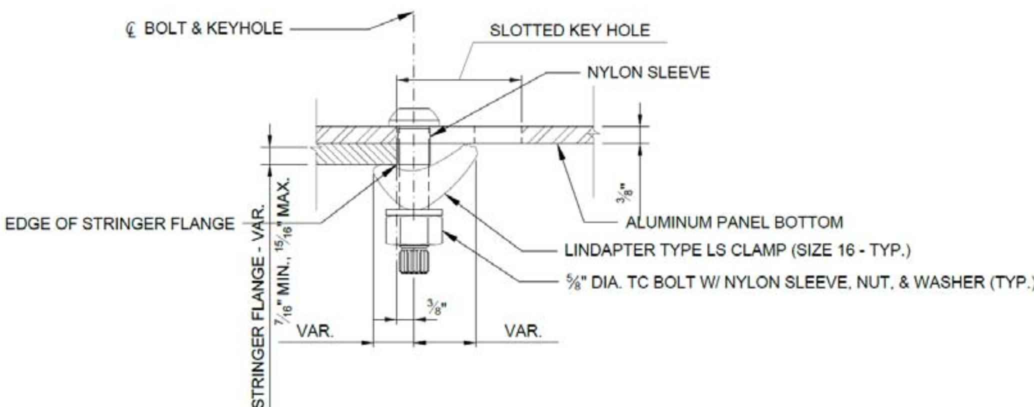

Figure 17. Commercial Clamp Detail (Bridgestone Americas Tire Operations Factory)

ASTM F3125 Grade F1852 (A325 TC) bolts are recommended with the clamps for the ability to tighten the bolts from the deck underside without need to hold the head within the hollow extrusion. The TC bolts are tensioned to a fraction of the full proof load (15 percent for a 5/8-inch diameter bolt and 10 percent for 3/4-inch diameter bolt which equates to approximately 2.8 kips), thereby leaving the bolts spline intact. The limited tension value is used to draw and hold the parts together without overloading the clamp. Thread locking adhesive (recommended) or other means is applied to prevent the nuts from loosening under vibrations. The length of commercially available blind type fasteners such as Lindapter Hollo-bolts and huck bolts typically do not accommodate the grip length. Zinc plating used on these fasteners is not as effective in protecting the steel hardware from galvanic corrosion compared to the mechanically galvanized coatings per ASTM B695 Class 50 used on TC bolts. Zinc plating has much less zinc than that of mechanically galvanized coatings, and thus is exhausted much earlier. Blind fasteners all require direct contact between the steel and aluminum and do not provide a means of physical separation to prevent galvanic corrosion other than the zinc coatings. TC bolts can be installed with nylon sleeves around the shanks and nylon washers or heavy-duty nylon tape under the head that separates the steel from aluminum. In addition, heavy duty nylon tape can be provided on the underside of the aluminum deck where galvanized steel clamps bear against the deck. The mechanically galvanized coatings on TC bolts and clamps provide a secondary level of protection from galvanic corrosion in the event of incidental contact.

A key hole detail in the bottom of the deck was developed to simplify bolt installation and avoid delivery of the bolt inside the voids, along the length of the extrusion. This became necessary as the clamps added grip length, making the bolts too long to fit within the limited height of the voids. Key holes consist of a larger hole at one end of sufficient diameter to accommodate the head of the bolt and a slotted hole to permit the shank to be shifted over to the specified location. The key hole can be fabricated by first coring the larger hole and then making the slotted hole with a router. As the connection is typically located in a region subject to compression the key holes do not pose a fatigue concern. A continuous deck over the stringer is typically in negative flexure with the bottom of the deck in compression and the discontinuous end of the deck is simply supported with zero or very low flexure. Stringers framed between floorbeams are typically considered simply supported with the top flange in compression.

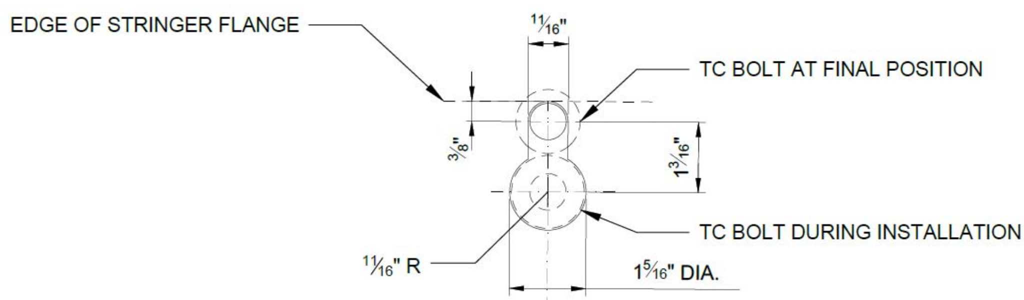

Figure 18. Key Hole Detail for 5/8-inch Diameter Bolt at Stringer Connection

With the relatively low clamping force and relatively large horizontal shear developed from live load and restrained thermal movement, slip between the deck and stringers is anticipated. To accommodate slip and prevent fretting of the protective steel coatings at the faying surfaces between the aluminum deck and steel stringer flanges, a thin (1/16-inch to 1/8-inch) elastomeric pad is recommended between the deck and stringer flange. The elastomeric pads are retained by the clamps on each side of the stringer flange and, if preferred, can be further secured with a suitable adhesive applied between the pads and underside of the aluminum deck and/or top of the stringer flange. In addition, slip at the clamp assemblies must also be considered. Friction between the galvanized steel clamps and steel stringer flange is greater than the friction between the clamps and aluminum deck with heavy duty nylon tape (e.g., 7 mil 3M High Temperature Nylon Tape 8555) cut to size and applied to the deck underside. Similarly, this friction is greater than the friction between the galvanized steel TC bolt and aluminum deck with a nylon washer or heavy duty nylon tape applied to the head of the TC bolt. As such, slip of the clamp assembly will occur between the clamp and aluminum deck and TC bolt and aluminum deck, rather than between the clamp and stringer flange, and thus fretting of the steel coatings is not anticipated. With the series of discrete deck panels, with a maximum width of 14 feet, slip movement is relatively small. As the bolts do not pass through the stringer flange, incidental bearing of the bolts against the hole does not generate significant horizontal shear in the bolts.

The clamp-type connection introduces the opportunity to reuse existing stringers, as the panels do not need to be pre-bolted to stringers in the shop, as previously recommended. The greatly simplified bolting details are anticipated to have a significant benefit in reducing deck installation cost.

Summary

This paper has outlined the clear advantages of aluminum orthotropic deck over steel open grid deck, providing similar weight, but without the historical issues with premature fatigue failures, attachment weld cracking, limited skid-resistance, and lack of protection of the supporting steel framing. The more durable aluminum orthotropic deck avoids periodic nuisance repairs and frequent deck replacement, like that typically required for steel open grid deck. With the noted positive results from laboratory and field testing, latest advancements and details that simplify fabrication and installation, lower life-cycle costs, and fewer impacts to the travelling public, bridge owners can now confidently replace aging steel open grid deck with aluminum orthotropic deck.