TENNESSEE'S EXPERIENCE WITH HIGH PERFORMANCE STEEL: AN OWNERS PROSPECTIVE

By Edward Wasserman and Henry Pate

High Strength, Quenched and Tempered Steels with 100 ksi yield strength have been available for about 35 years, originally sold under the trademark T-1 Steel, developed by US Steel. The first use known in a bridge was 1962. Fabrication of the second Carquinez Narrows Bridge over the San Joaquin River on I-80 made selective use of T-1 Steel in the trusses. Shortly afterward, in 1963 Tennessee followed suit with use in the Pennington Bend Bridge over the Cumberland River in Nashville, a plate girder bridge. In 1969 High Strength Quenched and Tempered Steels, ASTM A514 and A517 were introduced into the AASHTO Standard Specifications.

While Tennessee and other agencies have constructed successfully performing bridges with ASTM A514 and A517 steels, their susceptibility to hydrogen induced cracking made fabrication an expensive and strenuously controlled process. High values of heat input, post-weld treatment and strict control of welding consumables led to high costs of fabrication.

In response to the undesirable limitations, the Federal Highway Administration sponsored a cooperative effort between the American Iron and Steel Institute, the American Welding Society and the US Navy to collectively design and produce two High Performance Weathering Steels in 70 and 100 ksi grades.

As a result, HPS 70 W steel became available for use in early 1996 for fabrication and testing in an actual bridge. It was desired to use this new steel exclusively to document thoroughly the fabrication process and evaluate cost effectiveness.

At that time, the Tennessee Department of Transportation was completing the design of its first steel bridge utilizing the newly adopted AASHTO Load and Resistance Factor Design Specifications For Highway Bridges, using ASTM A709, Grade 50W Steel. The Department agreed to re-design solely utilizing HPS 70W steel to its fullest capabilities.

The new State Route 53 bridge over Martin Creek was let to contract in September of 1996 and opened to traffic in February 1998.

Since that time, the Department has taken on the construction of two, larger structures, optimizing the economical use of HPS 70W by designing and constructing bridge girders consisting of both homogeneous sections of HPS 70W steel, both in quenched and tempered (Q & T) versions and thermo-mechanically controlled processed (TMCP) versions as well as hybrid sections composed of HPS 70W flanges with 50W webs.

This paper will present background on the design and fabrication of these structures, outlining lessons learned.

FIGURE 1
State Route 53 over Martin Creek

FIGURE 2
State Route 52 over Clear Fork River

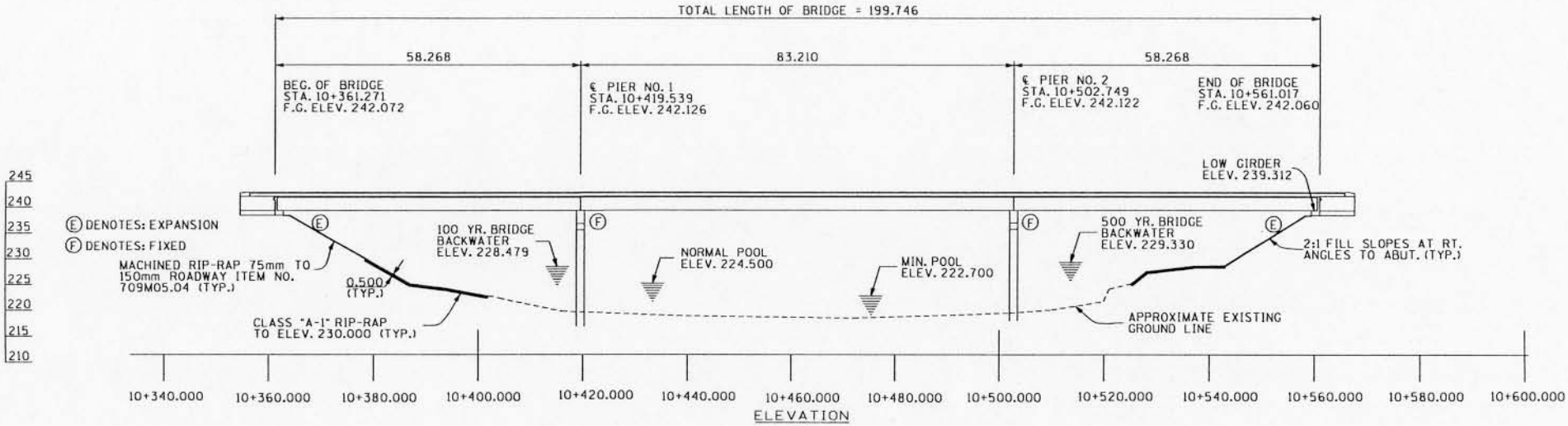

FIGURE 3
State Route 58/Clinch River

GENERAL DESIGN AND COSTS

The State Route 53 bridge over the Martin Creek Embayment is composed of two (2) 235.5-ft. spans carrying a 28-foot roadway on three continuous welded plate girders spaced 10-ft., 6-in. on centers (Figure I). The composite concrete deck is 30-ft., 4-in. wide and 8 ½-inches thick. The bridge is jointless, having integral, pile supported abutments. The one central hammerhead pier is 42-ft tall and rests, in turn, on a 24-ft deep seal footing, founded on rock.

All girder material is HPS-70W, while cross-frames are composed of AASHTO M270 grade 50W shapes. The webs vary from 7/16-in by 72-in in the positive moment region to ½-in by 72-in over the negative moment region. In the positive moment region, the top flanges are 18-inches wide and vary from 1-in to 1 1/8-in thick in two (2) increments while the bottom flanges, also 18-in wide, vary from 1-in to 2-in thick in four (4) increments. In the negative moment regions a doubly symmetrical section is used with flanges varying in four (4) increments of 11/8 in by 18-in., 1 5/8-in by 24-in and 2 ½-in by 24-in and 2 ½-in by 30-in.

The original design utilized 675,319 pounds of grade 50W steel estimated to cost $1.00 per pound. The re-designed structure contained 472,749 pounds of HPS-70W steel and 38,245 pounds of grade 50W steel with a bid price of $1.11 per pound. Two (2) subsequent supplements were granted to the fabricator, $25,000 to cover additional material shop splices due to plate length limitations (60-ft maximum) and $10,000 due to a change in welding flux (discussed later) which brought the steel price, erected, to $1.18 per pound. Therefore, using the HPS-70W saved 24.2 per cent in weight and 10.6 per cent in cost over the original design.

In the negative moment regions, the non-hybrid HPS-70W design was used to its best advantage. Grade 70 material requires a greater web thickness than Grade 50 in order to meet the minimum 2Dc/5tw ratios. However, because of the higher shear strength, the full bending capacity of the section could be used, since the actual shear stress in the webs did not exceed the allowable value, 0.6Vu. Further, in this design, there were no reductions in bending capacity due to the unbraced length of the compression flange. Another advantage of the all grade 70 section was a dramatic 31.4 per cent reduction in lifting weight.

In the positive moment regions an 18-inch flange was selected as the minimum width to allow for the option to use pre-cast, pre-stressed concrete deck forms. This width accommodated a 2 ½ inch overlap of the concrete panels plus the 3-inch strand projection without interfering with the two rows of shear studs used. The wider flange also stiffens the girder during erection and placement of the girder sections. The drawback to this wider flange is that the resulting b'/t ratio reduces the allowable compression stresses during erection and concrete slab pours. For the HPS 70W material, the nominal b'/t allowed can accommodate a 1-inch minimum thickness. The final compression flange thickness over a large portion of the positive moment section was controlled by the new constructability requirements found in Article 10.61 of the 1997 Interims to the Sixteenth Edition of The Standard Specifications, 1996. Under the provisions of Article 10.61:

fb = (26,200)(Alpha)(K)/D/tw2)Fyw (allowable stress)

Alpha = 1.3, for members without longitudinal stiffeners
D = depth of the web (inches)

tw = thickness of the web (inches)

k = 9(D/Dc)2, for members without longitudinal stiffeners

Dc = non-composite section web depth in compression (inches)

Article 10.61 requires that, under the wet concrete, the web may not exceed its elastic buckling limit. In other words, tension field action is disallowed in the slab pouring phase.

It should be noted that the allowable compression at this stage is controlled, not by strength of the web material, but by the geometry of the non-composite steel section. This requirement largely controls the size of the top flange and web thickness in the positive moment region. In the particular case of the girders of this bridge, the 18-inch compression flange was required to have a 1 1/8-inch thickness. The resulting compressive stress in the top flange, accounting for the dead load of the girder, wet concrete and construction loads, was 30.5 ksi. In this instance, a hybrid girder would have been a more economical solution, since shear stresses in this region are low, a lesser yield strength web could have been used with only a 5 per cent penalty in bending capacity. However, since tension field action would also be disallowed, a few additional stiffeners may have been needed.

There are no live load deflection limits in the LRFD Specifications. For this bridge the HS-20 Truck deflection met L/800, however the Lane Load did not.

Armed with the information gathered from our first project, the Department of Transportation initiated its second HPS 70W project, State Route 52 over the Clear Fork River (Figure 2). This bridge, situated in the Big South Fork National River and Recreational Area, is a 4-span welded plate girder structure with spans of 145/220/350/280 – feet. The superstructure is composed of four (4) girders spaced 12 – foot on centers, supporting a 42 – foot wide composite concrete slab. Rising over 200 – feet above the river, with only limited work space for cranes, weights need to be held to the minimum practical. Using HPS 70W steel helped achieve this.

With a length of 995 – feet a totally jointless bridge was not practical. Further, abutment number 1, on the low end of the 2.24 percent grade was to be founded on rock. Pier number 1,63 – foot in height and only 145 – feet from abutment 1 was determined to be easily deflected 1-3/8 – inches, the maximum possible displacement, without any significant moment addition at its base. Similarly, piers 2 and 3, with column heights of 183 and 124 – feet respectively could also deflect the maximum amount based on thermal movement accumulation presuming abutment number 1 as fixed. Calculations also indicated that the force developed by gravity loads acting on

the frictional resistance of sliding bearing surfaces would exceed the force required to deflect the piers. Accordingly, it was decided to design the bearings at abutment 1 and all piers as fixed. Expansion bearings and a roadway expansion device were placed only at abutment number2.

Generally speaking, in spans up to 350 – foot transversely stiffened girders are more economical than transversely and longitudinally stiffened girders. Further, optimization is achieved when the depth to thickness ratio (D/t) is at the maximum allowable limit. In this case a 96 – inch deep, ¾ - inch web was chosen for the non-hybrid HPS 70W section, over piers 2 and 3 as tensile and compressive bending stresses in the web approach yield. In the positive moment section, analysis is required to determine the minimum allowable thickness of the web. Due to the restrictions imposed by the constructability requirements of Article 10.61, coupled with the addition of the affects of adding the composite slab, as specified in Article 10.50 (b), the cumulative compressive stresses in the grade 50W web of the 350 – foot span only approached 42 ksi at ultimate load. Substituting the actual compressive web stress into the basic equation, D c/t = 36,500 / Fy ½ from Article 10.49.3.2, yields an allowable D c/t ratio of 178. Therefore the 96 – inch grade 50W web needed to be only ½ - inch.

Span 1, and Span 2 up to the dead load inflection point near Pier 2, were designed as non-hybrid using grade 50W steel. The negative moment sections at piers 2 and 3 were designed solely of HPS 70W, while the positive moment sections of spans 3 and 4 were designed as hybrid girders, utilizing grade 50W webs and compression flanges while using HPS 70W in the tension flanges. All stiffeners and cross frame material are grade 50W.

Utilizing HPS 70W for the pier sections at piers 2 and 3 reduced their lifting weights 30 percent over a grade 50W design. The overall cost of the girder for the Clear Fork River bridge, in place, was $1.03 per pound vs $1.18 per pound for the Martin Creek bridge even though the girders were fabricated at the same plant, shipped further and were more difficult to erect. Stock piled prices for HPS 70W steel for both bridges was $0.38 per pound while erection and profit at Martin Creek was $0.33 per pound and only $0.21 per pound at the Clear Fork River Site.

The Clinch River Bridge is a 3 – span continuous welded plate girder bridge with spans of 191/273/191 – feet (Figure 3). The 88 – foot wide slab is supported on Seven (7) girder lines, spaced 12 – foot, 9 – inches on centers. The negative moment pier sections were designed as non-hybrid using HPS 70W. The 24 – inch wide flanges, varying from 1 ½ to 2.5 – inches will be quenched and tempered, while the 9/16 – inch by 84 – inch web will be HPS 70W Thermal Mechanically Controlled Processed plate (TMCP), produced by Bethlehem – Lukens. This afforded the first opportunity to obtain fabrication experience with this new HPS material which does not require tempering and can, therefore, be furnished in longer plates, to reduce the number of butt splices required. Costs for the HPS 70W Q and T material, from the mill, were 44 cents per pound while the TMCP material was 40 cents per pound.

FABRICATION

The fabrication phase for the State Route 53 bridge started out inauspiciously. Prior to commencement of production welding, the Standard Road and Bridge Specifications of the Tennessee Department of Transportation and the AWS Specifications, required the fabricator, Trinity Industries, to perform weld procedure qualification tests on the steel by making several butt welds. Trinity made these samples using tandem LA100 wires and Lincoln Electric Company 880 M flux, supplied in bags. With this combination the test samples failed, in that an all weld metal sample broke during the elongation and tension test, due to hydrogen cracking. All other tests, (side bend, impact and tension on the base metal/weld metal samples) passed.

Because of the failure of the weld metal, Trinity requested a meeting, which was attended by Lincoln Electric representatives, Bob Nickerson of NBE Limited, consultant to the State and FHWA, Bill Wright, FHWA Project Manager, Dr. Yoni Adonyi, Professor of Le Tourneau University, and Mr. Bill Greer, TDOT Civil Engineering Manager in charge if Q/A Shop Inspection. After much discussion, Lincoln Electric Company representatives suggested Trinity Industries try the tandem LA100 wires with Mil-800H flux, shipped in sealed containers, along with a pre-heat of 250 degrees and an interpass temperature of 400 degrees F. They also suggested that the weld procedure qualification sample be left in restraint for 1 week. This latter suggestion was to restrain the sample transversely to the weld while it cooled, a very severe condition. If the sample did not crack, which it did not, one could have utmost confidence in the weld. The sample plates welded were 36-inches wide allowing for two samples. This was fortunate in that the first nominally all weld metal sample failed, yielding at 55 ksi, but reached an ultimate strength of 110 ksi. It was later discovered that the sample had been taken off-center of the weld and contained base metal from the heat affected zone (HAZ). A new sample was correctly taken, which passed, reaching yield at 101 ksi and ultimate strength at 110 ksi. Additional coupons from the weld prequalification test sample were removed by Dr. Adonyi and taken back to Le Tourneau for further testing. All further samples passed and fabrication was completed using 250°F pre-heat and 400°F interpass temperatures, without further incident. As mentioned earlier, using Mil-800H flux increased the project cost by $10,000.

At FHWA's direction, twelve samples utilizing various combinations of wire, flux and temperatures were prepared for further testing at Lehigh University. The combination samples were:

Three samples:	LA 100 wire, with 880M flux in bags, with no pre-heat
	LA 100 wire, with 880M flux in sealed containers, with no pre-heat
	LA 100 wire with Mil-800H flux in sealed containers, with no pre-heat
Three samples:	LA 100 wire, with 880M flux in bags, with 200 degrees F pre-heat
	LA 100 wire with 880 flux in sealed containers, with 200 degrees F pre-heat

LA 100 wire with Mil-800 flux in sealed containers, with 200 degrees F pre-heat.

Three samples: LA885 wire, with 880M flux in bags with 200 degrees F pre-heat.

LA885 wire, with 880M flux in sealed containers, with 200 degrees F pre-heat.

LA885 wire, with Mil-800H flux in sealed containers, with 200 degrees F pre-heat.

Three samples: LA885 wire, with 880M flux in bags, with no pre-heat.

LA885 wire, with 880M flux in sealed containers, with no pre-heat.

LA885 wire with Mil-800H flux in sealed containers, with no pre-heat.

The experiences with the State Route 53 over Martin Creek bridge contributed significantly to the recommendations contained in the Guide for Highway Bridge Fabrication with HPS 70W Steel. This document, funded and developed by the American Iron and Steel Institute (AISI), was adopted by AASHTO is a supplement to the ANSI/AASHTO/AWS Bridge Welding Code.

State Route 52 over the Clear Fork River was fabricated utilizing the Guide with the result that there were no fabrication anomalies.

For the State Route 58 over the Clinch River all butt welds for the HPS 70W flanges and webs were made using Lincoln LA85 electrodes in combination with Lincoln Mil800 HPNi flux. Importantly, in the joining of the HPS70W flanges to webs, undermatched welds were used. These single pass fillet welds were made with Lincoln L61 electrodes in combination with Lincoln 960 flux. This is in accordance with the Bridge Welding Code and is technically achievable because, while the yield strength is less than the yield strength of the HPS 70W material, the strain rate of the L61 weld metal is greater than that of the HPS 70W.

OVERALL ASSESSMENT

The Tennessee DOT is very pleased to have participated in demonstrating the viability ease and advantages of using HPS 70W steels for bridges. We are completely satisfied with the three bridges constructed to date. The use of HPS 70W will now be routinely used in all future projects, where they are appropriate.

Currently, the Department is designing a fourth project that will be utilizing HPS 70W TMCP webs and flanges, and if available, will utilize HPS 50W steel currently under development as well.