Proceedings

Advanced Materials for Construction of Bridges, Buildings, and Other Structures III

Engineering Conferences International *Year* 2003

The Evolution of Best Practices with High Performance Steel for Bridges

Edward P. Wasserman* W. Henry Pate† Tim Huff‡

*Tennessee Department of Transportation
†Tennessee Department of Transportation
‡Tennessee Department of Transportation

THE EVOLUTION OF BEST PRACTICES WITH HIGH PERFORMANCE STEEL FOR BRIDGES

Edward P. Wasserman[1]
W. Henry Pate[2]
Tim Huff[3]

ABSTRACT

High Performance Steel, grade 70 (HPS-70W) became available for use in early 1996 for fabrication and testing in bridges. Two (2) states, Nebraska and Tennessee agreed to be the first to implement usage. This paper provides a discussion of 3 Tennessee case histories in which high performance steel has been used to achieve weight and cost economies.

INTRODUCTION

Developments regarding High Performance Steels are continuing to evolve at a steady pace. As of this writing, over 115 bridges, across the United States, are either under construction or open to traffic and another 60 bridges are under design, according to the American Iron and Steel Institute (AISI). As a consequence new ideas, some good, some mis-informed, but all in need of comment are emerging.

Further, intense research on several fronts, directly and indirectly addressing HPS design is underway.

This paper will address some of these recent improvements (1) in the context of three (3) bridges already constructed in Tennessee:

1. State Route 53 over Martin Creek
2. State Route 52 over the Clear Fork River
3. State Route 58 over the Clinch River

[1] Director, Structures Division – Tennessee Department of Transportation, 505 Deaderick Street, Suite 1100, Nashville, TN 37243.

[2] Engineering Manager, Structures Division – Tennessee Department of Transportation, 505 Deaderick Street, Suite 1100, Nashville, TN 37243.

[3] Structural Supervisor, Structures Division – Tennessee Department of Transportation, 505 Deaderick Street, Suite 1100, Nashville, TN 37243

State Route 53 over Martin Creek

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 1). The composite concrete deck is 30-ft., 4-in. wide and 8½-inches thick. The bridge is jointless, having integral, pile supported abutments. The hammerhead pier is 42 ft tall and rests 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 $^{1}/_{2}$-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 1 $^{1}/_{8}$-in. by 18- in., 1 $^{5}/_{8}$-in by 24-in., 2 $^{1}/_{2}$ in by 24-in. and 2 $^{1}/_{2}$ –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 an erected price of $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. 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. The final compression flange thickness over a large portion of the positive moment section was controlled by the constructability requirements.

State Route 52 over the Clear Fork River

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 Recreation 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 workspace 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 heights were 63 ft, 183 ft and 124 ft. Bearings at abutment 1 and all piers were designed as fixed. Expansion bearings and a roadway expansion device were placed only at abutment number 2.

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 stiffener and cross frame material was 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.

State Route 58 over the Clinch River

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 ^{1}/2-inches are be quenched and tempered, while the 9/16-inch by 84-inch web is 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. Cost for the HPS-70W Q and T material, from the mill, was 44 cents per pound while the TMCP material was 40 cents per pound.

In the joining of the HPS-70W 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 (2) 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.

NEW DEVELOPMENTS AFFECTING HPS USAGE

Some new areas of research have been of great interest to bridge engineers. These include the development of TCMP HPS-50W steel, tension-field action considerations, and the development of HPS-100W steel.

TCMP HPS-50W Steel

Due to many requests from owners a new TCMP HPS-50W material, having the same chemistry as HPS-70W, has been developed and will be available as hot rolled plate in thickness of up to 4-inches. Steel material specifications conforming to HPS-50W have recently been added to ASTM A709-01, but due to procedural requirements the ASTM 709-01 specifications will not be included in the AASHTO M270 Specifications for another 12 to 18 months. Designers and owners should seriously evaluate where HPS-50W is specified, as it will, for the near term, cost about $0.05 per pound more than ASTM A709 50W material. Best usage would be for webs and flanges in tension only. The customary 50W is recommended for compression flanges, stiffener material, and web bolted splice plates.

Tension Field Action Considerations

The use of HPS-70W in combination with A709-50W to produce hybrid girders has proved to be the most popular application for the new steel. However, it was recognized that even more economies could be realized if the present restrictions on shear designs for hybrid girders could be removed. Presently, the AASHTO Standard Specifications, and LRFD Specifications limit the shear strength of webs to the elastic-buckling limit, dis-allowing tension field action (Figure 4). The basis for the prohibition is that no tests on hybrid girder shear capacity had been conducted. The obvious concern has been that when the higher strength flanges reach yield, the lower strength web material will have gone beyond yield, thus shedding additional stress to the flanges (Figure 5). Based on this position, more stiffeners are required to satisfy code requirements.

Research into hybrid girders by Dr. Michael Barker at the University of Missouri, Dr. Don White at Georgia Tech, and Dr. Atorod Azizinamini at the University of Nebraska-Lincoln has been completed. The results demonstrate that hybrid girders can exhibit tension field action in magnitudes consistent with homogeneous girders. Further, results indicate it is possible to remove moment-shear interaction considerations for braced non-compact sections. In June of 2003 the AASHTO Subcommittee on Bridges and Structures approved a rewrite of the LRFD Specifications which included the elimination of hybrid girder considerations in certain cases. These changes are applicable to girders with web yield strength at least 70% of the flange yield strength. Consideration of shear-moment-interaction has been eliminated for this class of hybrid girder.

HPS-100 Steel Usage

HPS-100W plate material can currently be produced by Bethlehem-Luken's successor, International Steel Group. This relatively new material is based on Cu-Ni alloying and 0.6 percent carbon content. Development of HPS-100W steel has been carried out in conjunction with Lehigh University.

Even thought HPS grades of 100W are now available, the problem of hydrogen cracking in currently used matching weld consumables still exists. It appears that the development of welding procedures has not kept pace with the development of the new materials.

Alternatives that could result in an effective use of HPS-100W steel include the use of under-matching weld material in butt welds. Dr. Alan Pense and Nicole Repetto at the ATLSS Center of Lehigh University have conducted experiments that build on previous studies by the Japan Welding Engineering Society as well as previous work at Lehigh. The work focuses on the use of 80 ksi weld metal joining 100 ksi plate material. Work to date has concluded that:

1. HPS-100W plate with width-thickness ratio of 7 can develop full tensile strength with 8.5% overall ductility when the weld metal strength is within 5.5% of the base metal yield and 3.5% of the base metal tensile strength.

2. HPS-100W plate with width-thickness ratio of 7 can develop yield strength with 2.5% overall ductility when the weld metal strength is within 21% of the base metal yield and 20% of the base metal tensile strength.

3. HPS-70W plate with width-thickness ratio of 7 can develop full tensile strength with 6.0% overall ductility when the weld metal strength is within 9.3% of the base metal yield and 13% of the base metal tensile strength.

4. HPS-70W plate with width-thickness ratio of 16 can develop full tensile strength with 6.0% overall ductility when the weld metal strength is within 26% of the base metal yield and 20% of the base metal tensile strength.

As a safety measure, an additional strategy would be to locate these welds at locations where the demand has decreased to correspond with the yield strength of the weld metal.

PRACTICES NEEDING RE-EVALUATION

Several projects let to contract reportedly have contained specification requirements that, on face value, appear to not have been fully rationalized.

- As is well known, HPS-70W steels easily meet the minimum toughness requirements of zone 3 Charpy impact values. Some projects, for bridges, in zones 1 or 2, are requiring that all Grade 50 material in tension flanges and webs also meet zone 3 charpy values. This only leads to increased costs for the grade 50 material for the sake of over conservatism. Specifying HPS-70W implicitly obtains zone 3 charpy quality. However, plans should specify only the zone appropriate for a bridge's geographic region.

- Some projects have required that filler plates, used in HPS-70W field splices to be HPS-70W. This is unnecessary. The filler plates could be Grade 50W or ASTM A606 weathering grade sheet material and serve adequately, and need not meet charpy requirements.

- Some projects have called for HPS-70W web splice plates to have Charpy values in both the longitudinal and transverse directions, which, again is overly conservative. Stresses in web sections for bolted splices are dominated by shear loads, not moment. Further bolted splices are redundant. In the opinion of the Tennessee DOT, web splice plates need not require charpy quality.

CONCLUSION AND SUMMARY

As can be seen issues dealing with HPS steels remains in a dynamic state, with new products and new information on the mechanical properties coming to light. These events are expected to continue for sometime, making use of these improved steel even more versatile and economical.

To summarize the findings:

1. HPS-70W steel can be used to achieve both weight and cost savings in long span bridges. The most effective use appears to be in negative moment regions and tension flanges of positive moment regions.

2. AASHTO has taken into account research into hybrid girder reductions and eased these requirements in its latest LRFD Specification. This could result in further savings.

3. The ability to use HPS-100W steel in a cost-effective manner in beam-type bridges is an item yet to be determined.

4. HPS-70W steels provide plastic rotational capabilities comparable to grade 50W steels

5. Utilize weathering properties of HPS 70W (HPS 485W) and paint only the abutment ends of girders

6. Use 50W (345W) steel where practical within a girder.

7. Use all HPS 70W (HPS485W) over interior supports, where shears <u>and</u> moments are high and design is non-composite.

8. Use hybrid sections in positive moment sections where moments are high but shears are low.

9. Allow under-matching fillet welds where permissible to reduce cost of weld consumables and fabrication.

10. Be aware of length limitations for quenched and tempered plates and plan cut-offs as well.

11. Consider waiving live load limits for lane load deflections.

12. Utilize TMCP HPS plate where possible.

13. Field splice filler plates for HPS 70W girders can be 50W steel.

14. Web splice plates need not meet Charpy quality requirements.

15. HPS 70W steels easily meet Charpy toughness requirements for zone 3 impact. This does not mean that the 50W plate use along with HPS 70W in a hybrid girder has to meet zone 3 requirements. The 50W steel only need meet the appropriate zone requirements, whether 1, 2, or 3.

16. Use the <u>Guide Specification For Highway Bridge Fabrication With HPS 70W Steel</u> (<u>4</u>) and <u>High Performance Steel Designer's Guide</u> (<u>5</u>).

REFERENCES

1. Wasserman, E. P. and Henry W. Pate, "Tennessee's Experience with High Performance Steel: An Owner's Perspective", Proceedings – Steel Bridge Design and Construction for the New Millennium, FHWA/National Bridge Research Organization, Lincoln, Nebraska, December 2000.
2. American Association of State Highway and Transportation Officials (AASHTO) and American Welding Society (AWS), joint publication D1.5-96, Bridge Welding Code, July 1996.
3. American Association of State Highway and Transportation Officials (AASHTO), Standard Specifications for Highway Bridges, 17th edition, 2002.
4. AISI-FHWA-Navy HPS Steering Committee and Welding Advisory Group, Guide Specification for Highway Bridge Fabrication with HPS 70W (HPS 485W) Steel, 2nd edition, June 2003.
5. Lwin, M. Myint, High Performance Steel Designer's Guide, FHWA Western Resource Center,2nd edition, April 2002.

Figure 1. State Route 53 over Martin Creek

Figure 2. State Route 52 over the Clear Fork River

Figure 3. State Route 58 over the Clinch River

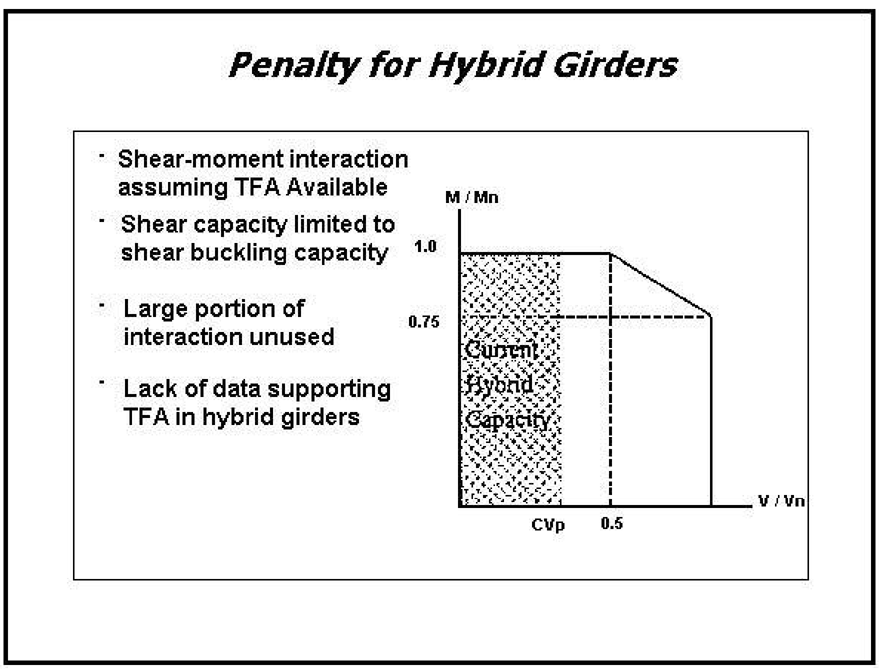

Figure 4. Hybrid Girders and Tension-Field Action

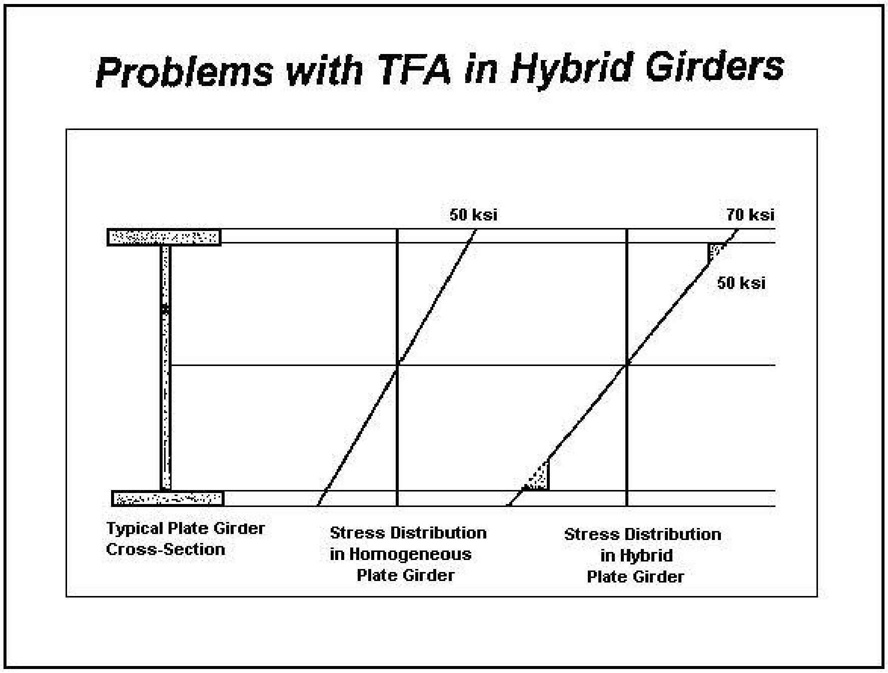

Figure 5. TFA Concerns

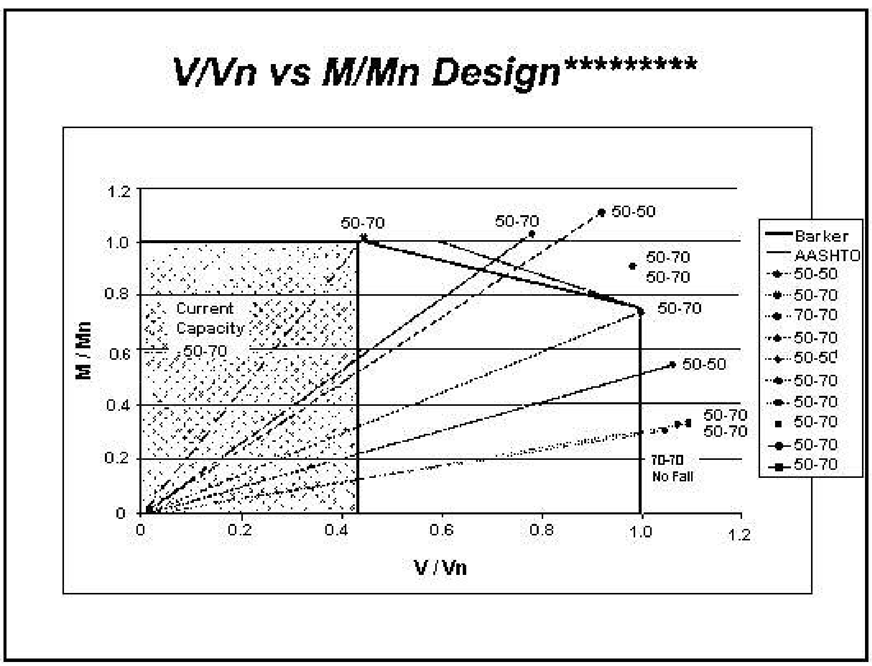

Figure 6. Moment-Shear Interaction

KEY WORDS

bridge
steel
ductility
tension-field action
welding
hybrid
girder
AASHTO