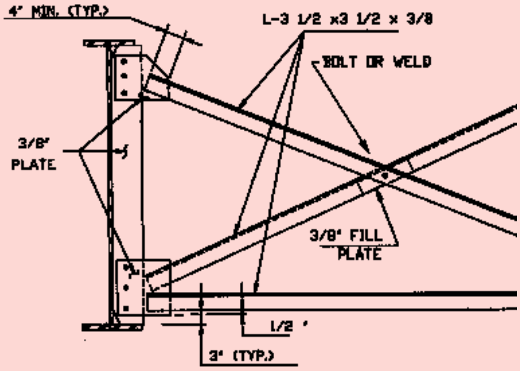


BRIDGE CROSSINGS

No. 15, October 1999

Practical Information For The Bridge Industry

Nebraska's Three-Phase High Performance Steel Bridge Initiative

By Michael W. Beacham, P.E.

While higher strength steel is often sought in bridge design, past efforts have been hindered by questions about its weldability. Now, thanks to a cooperative research program initiated in 1994 between the U.S. Navy, the Federal Highway Administration (FHWA), and the American Iron and Steel Institute (AISI), a new grade of steel is available with enhanced performance characteristics.

Commercially available in 1997 HPS-70W (HPS-485W) has a lower carbon content than conventional 70-ksi steel, which results in substantially improved weldability. And as an added bonus, since HPS-70W is a weathering steel, painting is unnecessary. Finally, HPS-70W may require no preheat during welding, a characteristic that makes field welding a possibility. At the present, however, the recommendation is to use preheat until ongoing research projects are completed.

Limitations On Use

As with any new material, its use initially is subject to limitations imposed by the governing design codes. For example, when high strength concrete became popular in late '80s, the ACI 318-89 code imposed several limitations on design provisions. (In the case of high strength concrete these limitations were in the form of limiting the concrete compressive strength.)

Code-imposed limitations usually reflect the lack of research data rather than the inability of the material to resist loads. Since most design provisions are based on experimental tests and are empirical, a substantial body of research data is needed before codes will incorporate new materials such as HPS-70W. As a result, the 16th Edition of the AASHTO Standard Specification and the AASHTO LRFD Bridge Design Specifications, contain several limitations that prevent taking full advantage of the higher yield strength of HPS-70W. Most of these limitations are related to strength capacity of plate girders in the positive and negative bending moment regions. Some of these limitations are as follows:

- Positive section capacity of continuous plate girder bridges with non-compact interior section could be limited to M_y (yield moment capacity of the section), even if the positive section meets the compact section and ductility requirements.
- Negative section capacity of compact sections are limited to M_y, instead of Mp (plastic moment capacity).
- The 10% redistribution of negative moment for continuous plate girders with compact interior sections is not allowed.
- Ductility requirements for composite sections in positive bending moment regions requires a beta factor that is provided for 36 and 50-ksi, but not for HPS-70W steel.

The good news, however, is that research studies sponsored by National Steel Bridge Alliance (NSBA), AISI, and FHWA were initiated in 1996 to resolve some of these design related questions. Three institutions were involved in initial research studies, University of Nebraska at Lincoln (UNL), Georgia Institute of Technology and Lehigh University. Results of these ongoing

Pictured above is a creek crossing on Highway 79 south of Snyder, NE. The crossing is Nebraska's first use of the new HPS-70W material.

studies have provided information to formulate design recommendations that could eliminate some of these limitations. For the moment, the main conclusions is that for plate girders HPS-70W could attain plastic moment capacity; however, the 10% redistribution moment is not allowed. Additionally, the inelastic methods of analysis and design are not permitted when HPS-70W is used.

One of the limitations in the AASHTO code that is expected to influence the economy of high performance steel bridges are limitations on hybrid sections. AASHTO code does not allow taking advantage of tension field action for hybrid sections, when considering shear design. When plate girders with slender webs are provided with intermediate stiffeners, the shear capacity of the panel between intermediate stiffeners could be increased beyond the shear that will cause elastic buckling. This increase in shear capacity is allowed because of formation of "tension field" action following the web elastic buckling. When hybrid sections are used, this additional shear capacity due to tension field action is not allowed. The limited experience with high performance steel indicates that the most economical plate girder could consist of HPS-70W flanges and a lower strength web material such as 50-ksi steel. Such plate girders are hybrid and according to current AASHTO code tension field action can not be used when calculating shear capacity.

Additional studies by Modjeski and Master and J. Muller International have developed several bridge configurations that are better suited for higher strength steels. Future steel bridges could potentially use shapes other than the traditional "I" shape plate girders. Introducing such changes, however, will perhaps be more challenging from a "construction culture" stand point than technical feasibility. The good news is that Tennessee and Nebraska initaives are not the only states to use HPS-70W in bridge construction. Several other states have initiated steel bridge projects where HPS-70W will be used. The sum of these activities is indicative of upcoming new and exciting horizons in steel bridge design and construction.

Nebraska Initiative

"Nebraska's Three Phase High Performance Steel (HPS) Bridge Initiative" is a case study to investigate the many advantages high performance steel offers in bridge construction. This case study will be implemented in three different though very closely related phases.

The work to be conducted involves the cooperation of FHWA, the Nebraska Department of Roads (NDOR), and the National Bridge Research Organization (NaBRO) at the University of Nebraska-Lincoln (UNL). NSBA, AISI and Lincoln Steel are also participants in the project. NDOR is the governing agency that oversees and executes contracts, design, and construction. UNL will be responsible for research aspects of the case study, instrumentation and monitoring of the bridges, and preparation and delivery of final documentation of all three phases of the case study. The FHWA Nebraska Division office will oversee the development of the project and provide continuous support and guidance to involved parties. Phase I is now complete, Phase II has begun and is scheduled for completion in early 2000 and Phase III began in May of 1999 and will be completed in late 2001.

In Phase I of the study, a 150' simply supported steel girder bridge was designed assuming 50-ksi material for the girders. However, actual fabrication of the girders used HPS-70W steel. This constitutes a direct substitution of HPS-70W steel for 50-ksi steel without taking advantage of the higher performance characteristics offered by HPS-70W steel. This approach helped to understand the fabrication issues of HPS-70W. Laboratory studies conducted by UNL during Phase I developed information addressing the current 50-ksi limitations given in the AASHTO design codes. Phase I also included conducting two regional seminars for state highway agencies, and full documentation of all Phase I activities.

The purpose of Phase II is to optimize the girder design using the higher yield strength HPS-70W. Phase II involves the design and construction of a two-span continuous steel bridge utilizing HPS-70W steel. The length of each span is approximately 236'. Findings from Phase I and other researchers were provided to NDOR bridge engineers to optimize the bridge design. NDOR provided optimized 50-ksi and HPS-70W design results for comparison. The constructed two-span bridge will be instrumented and monitored. One regional seminar for state highway agencies and full documentation of Phase II activities will also be carried out. Also during Phase II an innovative bridge configuration will be selected and evaluated for Phase III.

Scale model testing of the innovative bridge system and/or its components will be analyzed during Phase II to ensure its safety and compliance with AASHTO code requirements.

Phase III will involve the design and construction of the innovative bridge system selected in Phase II. The needed information for implementing Phase III of the project will be obtained during Phases I and II. This includes design of the new bridge system, testing of the model system and/or its components to ensure its safety, and compliance with AASHTO requirements. A key element of Phase III of the project will be conducting a regional or national seminar for state highway agencies and other interested individuals and full documentation of the tasks conducted in all three phases of the project.

Phase I

Eliminating the 50-ksi limitations was the main task of the testing program for Phase I. The 1996 version of AASHTO LRFD Bridge Design Specification (hereafter referred to as AASHTO code) limited the yield strength of the steel to 50-ksi in several sections (sections 6.10.2.1, 6.10.2.2, 6.10.5.2.2b, 6.10.6.2 and 6. 10. 11). These limitations are merely reflections of the lack of test data. The main concern is that as the ratio of yield to tensile strength increases the inelastic deformation capacity of steel girder sections may decrease. Specifically, the moment rotation characteristic of HPS steel needs to be verified. The testing program was intended to develop adequate information so that

50-ksi limitations can be eliminated where possible. The testing program compliments the tests conducted previously at UNL, Lehigh, FHWA and elsewhere.

For a continuous steel girder bridge, when the length of each span becomes larger than approximately 100' (a situation that is the case in Phase II of the project), the most economical system is to use a non-compact section over the pier. Therefore, the moment rotation behaviors of both compact and non-compact sections in the negative regions were studied.

Four specimens representing the negative bending moment regions were tested at UNL. Results of these tests demonstrated that HPS-70W girder sections, meeting the compact web and flange slenderness limits of AASHTO specifications and with lateral bracing meeting the plastic limit state requirements, could reach their plastic moment capacity. However, these sections may not provide the rotational ductility of 3, as is currently implied by the AASHTO specifications. Therefore, considering the current rotational ductility demand of 3 for continuous plate girders, the 10% moment redistribution or the inelastic method of analysis and design should not be permitted for HPS-70W steels.

The test results obtained from non-compact HPS-70W specimens indicate that current provisions in AASHTO specifications are applicable to 70 ksi steels.

Two of the specimens tested at UNL had non-compact sections. One of these specimens utilized 50 ksi steel, while the other one used HPS-70W steel in its fabrication. The proportioning of these two specimens were such that their normalized results could be compared. Results of these two tests indicated that plate girders with non-compact sections, using 50 ksi steel or HPS-70W steel, have similar behavior.

In addition to the experimental program to comprehend the behavior of compact and non-compact HPS plate girders, an extensive numerical and analytical program is also underway at UNL. This numerical work is in the form of conducting series of non-liner finite element analyses. The objective of this program is to comprehend the flexural behavior of steel plate girders. Results of this work should be available within the next year.

Two simply supported composite test specimens have also been fabricated and tested to address the behavior of HPS composite plate girders in positive sections. Each specimen consisted of a single steel girder connected to a concrete slab over the top flange using shear studs.

Results of tests conducted on positive sections indicate that current AASHTO LRFD ductility requirements are very conservative and could be relaxed. In addition to conducting experimental tests to comprehend the behavior of the positive sections, analytical studies were also carried out to develop missing information in AASHTO specifications, with regard to extending the existing ductility requirements for HPS plate girders. This work has been completed and a beta factor of 0.7 has been suggested to be used for HPS-70W steel.

The provisions of section 6.10.11 of AASHTO LRFD are limited to 50-ksi steel and are applicable to compact sections only. The preliminary conclusion is that the provisions of section 6.10.11 of AASHTO LRFD, which recognizes the inelastic behavior of steel plate girders, should not be used in conjunction with HPS steel having a yield strength in excess of 50 ksi. The main reason for this preliminary conclusion is the lack of adequate rotational capacity for HPS-70W plate girders. AASHTO specifications imply that compact sections should demonstrate rotational ductility of at least 3. Tests conducted to date indicate that compact HPS-70W plate girders do not provide such rotational ductility. It should be noted that existing data indicates that 36 or 50 ksi steels may not provide such rotational demand either.

The entire fabrication process from the time of receiving the plate materials at the fabrication shop to completion of the bridge construction will be documented and provided to FHWA. This documentation will include slides at various stages of the productions and accompanying text.

Phase II

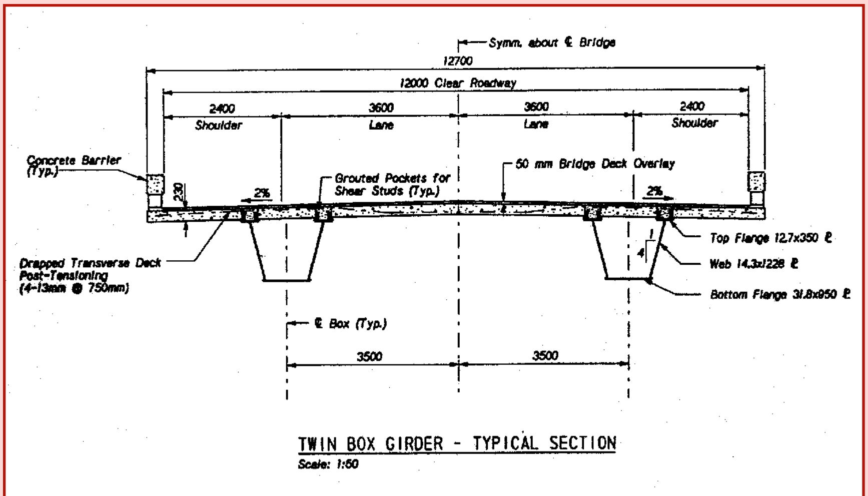
The major focus of Phase II is the design and construction of a two span optimized continuous steel girder bridge and to monitor its behavior under dynamic loads. NDOR designed the bridge for both 50 and HPS-70W steel. This will allow a differential design cost comparison to be made. Each span of the bridge will be approximately 236 feet with integral abutments. Both haunched and straight welded plate girder designs were considered for the design's optimization. Construction of the Phase II bridge began in April of 1999.

The Phase II bridge will be instrumented and monitored. For long-term behavior, vibrating wire gages, mechanical DEMAC points and pressure cells will be used. Vibrating wire gages will be embedded in slab concrete over the middle support, quarter points and mid spans of the bridge prior to casting slab. The vibrating wire gages will also be placed at the same locations along the depth of the girders. This information will be used to study creep and shrinkage behavior of the bridge during the curing process. Pressure cells will be used at abutments to gather information on thermal movements of the bridge. Short #3 reinforcing bars with attached electrical gages will be embedded in the concrete slab, over the middle support, quarter points, and mid spans of the bridge before casting the slab. This instrumentation will be used to conduct truck load tests, studying possible changes in bridge behavior over time. Pressure cells will be used to monitor force changes at the integral abutment locations due to seasonal temperature variations.

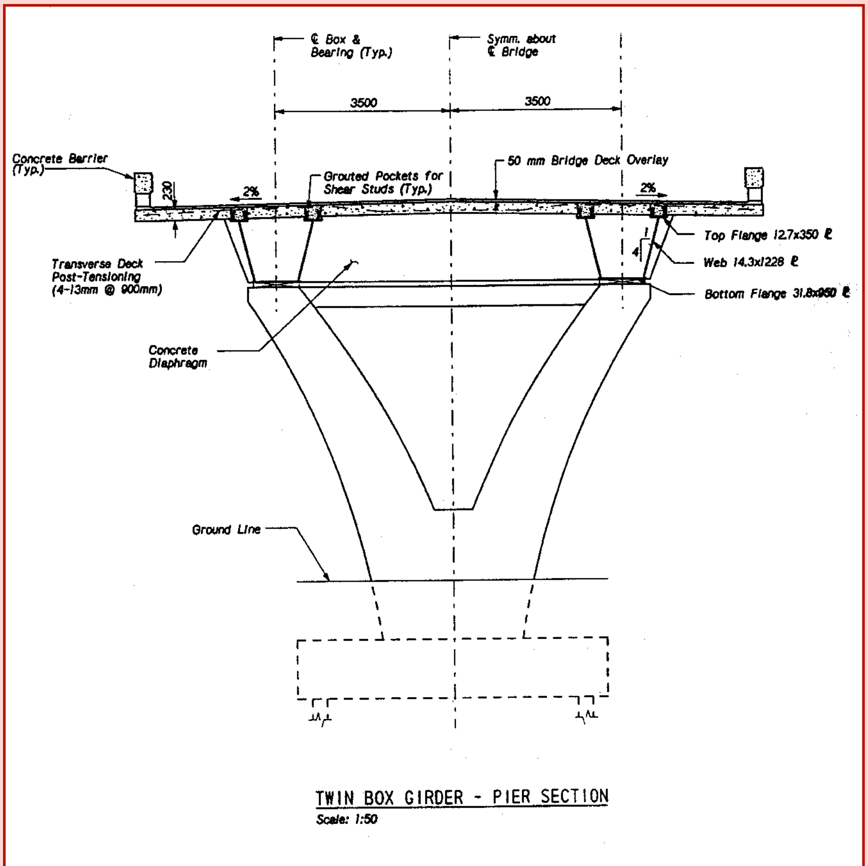
Control concrete specimens will be prepared during casting of the slab to develop free shrinkage characteristics of the concrete. This information will be used in conjunction with data collected from the bridge during the curing process to better estimate the shrinkage deflection of the bridge directly. A report published by the UNL on full scale testing of a steel bridge outlines a procedure that results in a very accurate estimation of deflection of steel bridges due to creep and shrinkage.

Instrumentation placed in the bridge before casting the slab will allow monitoring of the behavior of the bridge. This will be accomplished by testing the bridge after completion of the construction and before opening to traffic, six months after construction, and one year after construction. At these intervals, the bridge will be subjected to live loads by using one or two relatively heavy trucks traveling at various configurations and speeds over the bridge. Additionally, the bridge will be subjected to dynamic loading at the same intervals using

Phase III Innovative Concept



Phase III Innovative Concept



shakers placed at various places on the bridge.

During this phase of the study, information necessary to implement Phase III of the project will be developed. An appropriate model of the innovative steel bridge system will be designed, constructed and tested at the structural laboratory of the UNL to examine safety issues and establish behavior at various load levels. The scale of the model bridge will be determined based upon the maximum size of the structure that can be tested in the structural laboratory (120').

Phase III

The major focus of this phase will be the design, construction, and monitoring of an innovative bridge system. An outside consultant will carry out the design of the innovative system. In consultation with FHWA, NDOR and an outside consultant, an instrumentation plan will be developed to observe the behavior of the system at important locations within the bridge. The instrumentation of this bridge will be of extreme importance, since it will be a unique bridge type for which previous data do not exist. Therefore, the instrumentation and monitoring program for this bridge will be more extensive than the previous two bridges in Phases I and II.

The uniqueness of this innovative bridge will make it very important to extend the monitoring period beyond one year. Funding for monitoring the bridge beyond the first year will be sought from sources other than FHWA. At the appropriate time, a request will be submitted to NDOR for continuous monitoring of this innovative bridge.

The collected data will be analyzed continuously and compared to assumptions made during the design. Additionally, the collected data could also be used to make necessary changes to the design of similar systems in the future.

The entire fabrication process of the bridge in Phase III, from the activities in the shop to completion of the bridge construction, will be documented. This documentation will include slides at various stages of the fabrication with accompanying text. Phase III will be the key period for transfer of knowledge gained during all three phases of the case study.

Michael W. Beacham, P.E., is the NSBA's Director of Construction Services-Midwest and can be reached at 402/797-7101 (email: beacham@aiscmail.com).