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Structural Engineering Documents

8

Use and Application of High-Performance Steels for Steel Structures

International Association for Bridge and Structural Engineering IABSE
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Foreword

Due to their good material and fabrication properties High-Performance Steels (HPS) are finding an ever increasing use in structural applications, especially in bridge design. However, the development and the code requirements of the various countries and regions in the world differ markedly from each other.

With this background, the members of Working Commission 2 of the International Association for Bridge and Structural Engineering (IABSE), especially Prof. Joël Raoul from SETRA, Paris, had the idea of preparing a state-of-the-art document on the use and application of this new generation of steel grades. This proposal, supported by the chairperson of Working Commission 2, Prof. Dr Ulrike Kuhlmann, was the starting point for the preparation of this document.

In comparison to existing Structural Engineering Documents which were written by only one or two experts, this document includes contributions from a number of experienced international authors showing the worldwide development of High-Performance Steels.

I wish to acknowledge the support given to the preparation of this document and express my thanks to all contributing authors and especially to Prof. Dr Bernt Johansson and Prof. Dr Chitoshi Miki for their support and involvement in organizing the contributions from the various countries. I also want to thank Prof. Joël Raoul who had the initial idea and Prof. Dr Ulrike Kuhlmann who gave me the opportunity and time to coordinate the whole document. Many thanks are also given to Dr Geoff Taplin, Dr Sylvie Boulanger, Dr Tomonori Tominaga and Dr Roger Pope who have spent much time in reviewing and rereading the whole document.

Finally I would like to thank IABSE for the publication of this Structural Engineering Document.

Stuttgart, October 2005

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1 Introduction and Aim

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Over 15,000 tonnes of "High-Performance Steel" (HPS) were used to build the Millau viaduct in France (see Fig. 5.5.12 and 5.5.13), in order to satisfy the performance criteria of the design team. Steel grades of that category generally lead to cost reductions, smaller sizes of components, lightweight structures and less welding work. Most importantly, these new grades contribute to a sustainable environment due to improved durability properties and reduced material use. For medium and long span bridges, weight reduction can reach 20%.

"High-Performance Steel" (HPS) is the designation given to steels that offer higher performance in tensile strength, toughness, weldability, cold formability and corrosion resistance compared to the traditionally used mild steel grades. In the past fifteen years, there have been significant improvements in steel making technologies, both in terms of metallurgical advances, and rolling and heat treatment process developments. One of the important technologies in this context is the Thermo-Mechanical Control Process (TMCP) that adequately controls rolling and cooling within the steel plate production in order to generate fine microstructures. The TMCP technology has been instrumental in providing higher strength, better weldability and excellent toughness qualities. Only through these technological breakthroughs has it been possible to produce HPS for the construction industry.

The development of HPS goes a long way to address a new societal demand for slender lightweight structures for the design of medium to long span bridges and multistorey buildings. In such structures, there is a strong requirement to use high strength materials that can also meet erection and fabrication demands. HPS adequately fulfills these requirements leading to economical bridge and building structures with a great potential use for new effective and aesthetic structural solutions.

To encourage engineers to consider HPS in their designs, especially in the field of bridge construction, the deployment and sharing of specialized knowledge on this new steel grade was deemed essential. At the moment, current design codes do not contain sufficient guidelines to fully explore the properties of HPS. Hence, the scope of this document is to provide:

- information on the production process and its impact on steel quality,
- chemical composition and mechanical properties of HPS in terms of strength, toughness, weldability and corrosion resistance,

– design, fabrication and erection recommendations based on existing codes and research results,
– actual examples and technical solutions for various applications and
– a summary on the high-performance steel grades available in several countries.

The aim of the document is to provide an overview on the development and application of HPS at an international level. The document is not a monograph but an assembly of papers from different countries with a focus either on the material, the standard, or its use and application. Some of the papers have a research focus and others have a practical focus.
The design and application of HPS often needs new design philosophies and advanced structural and technical solutions. The enclosed examples of applications should thus help to give more details and references based on existing experience. The reader will note that most of the examples relate to bridge structures, where one can take full advantage of the improved properties of HPS, where reduction of weight is an important issue and deflection is in many cases not the governing factor in the design process. The document is divided into six chapters and includes North American, Japanese and European authors whose goals were to contribute a state-of-the-art review.

2 High-Performance Steels in the United States

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2.1 Introduction

In 1992, the U.S. Federal Highway Administration (FHWA) initiated an effort with the American Iron and Steel Institute (AISI) and the U. S. Navy (Navy) to develop new high-performance steels (HPS) for bridges. The driving force for this project was the need to develop improved higher strength, improved weldability, higher toughness steels to improve the overall quality and fabricability of steels used in bridges in the United States. It was furthermore established that such steels should be "weathering". By this is meant the ability to perform without painting under normal atmospheric conditions. The Timeline of the HPS program is shown in Fig. 2.1.1.

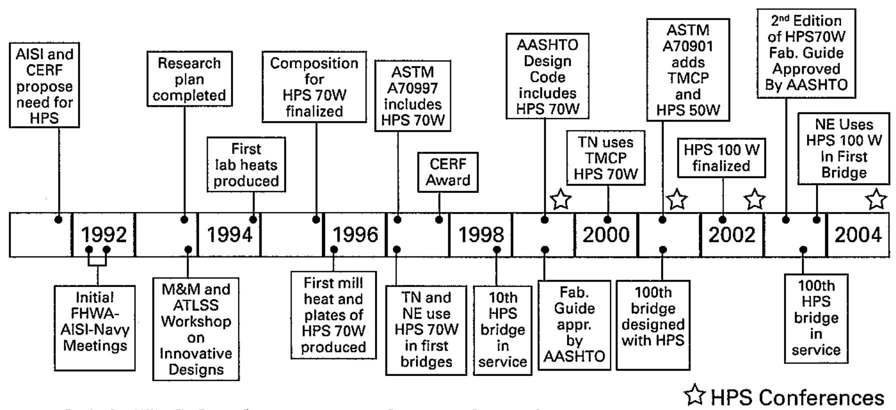


Fig. 2.1.1: HPS development timeline in the U.S.

In the United States, the principal steel specifications for bridges are American Society for Testing and Materials (ASTM) A709 and American Association of State Highway and Transportation Officials (AASHTO) M270. Currently, in these specifications, there are steel grades with minimum yield strengths (Y.S.) of 36, 50, 70, 100 ksi (250, 345, 485 and 690 MPa). These minimum yield strengths also serve as the grade identity. Furthermore, when the steel has a weathering capability, the letter "W" is attached to the grade number, for example, grades 50W and 100W. In the United States, Charpy V-Notch (CVN) impact testing requirements for bridge steels were developed by dividing the country into three zones. These zones range from the northern climates (known as Zone 3) to the southern climates (known as Zone 1) and have specific CVN testing temperature requirements. It was decided by the Steering Committee that the goal for HPS design would be to develop a steel that could meet the most critical requirements of Zone 3.

2.2 HPS Research and Development Program

A Steering Committee made up of representatives of the three noted groups, as well as fabricators, welding consumable suppliers and university representatives was established. They identified goals of the research program, which were to develop 50, 70 and 100 ksi (345, 485, 690 MPa) minimum yield strength, weathering steel grades, with toughness meeting Zone 3 requirements and significantly improved weldability [2.1]. The mechanical property goals were detailed in Table 2.2.1.

	HPS 50W up to 4" (101 mm)	HPS 70W up to 4" (101 mm)	HPS 100W up to 2.5" (64 mm)
Yield Strenght, ksi (MPa) minimum	50 (345)	70 (485)	100 (690)
Ultimate Tensile Strength, ksi (MPa)	70 min. (485)	85–110 (586–760)	110–130 (760–895)
CVN of 35 ft.-lbs. (48 J)	+10°F (-12°C) *	-10°F (-23°C)	-30°F (-34°C)
* 30 ft-lbs. (41J)			

Table 2.2.1: Mechanical properties of high-performance steels

These goals represent the strength level and the aggressive CVN toughness requirements felt important for these improved steels. The additional key goal of the program was to develop improved weldability steels, meaning that lower carbon and carbon equivalent levels would be required. Referring to Fig. 2.2.1, known as the Granville Weldability Diagram [2.2], new HPS steels were developed, which improved upon the traditional range of carbon and carbon equivalent levels for bridge steels.
The requirement that these new steels be able to demonstrate weathering characteristics in atmospheric environments meant that the current grades 70W and 100W should be used as a basis for the new grades. These grades contain elements such as Cr, Cu, Ni and Mo that are important to providing weathering characteristics. Improved weathering steels at all strength levels are of interest in bridges.
To provide improved toughness levels, a steel with lower sulfur level is required. It was decided that the goal for these grades would be a 0.006% maximum sulfur level with calcium treatment for inclusion shape control. This is a major improvement over traditional sulfur maximum of 0.05%.

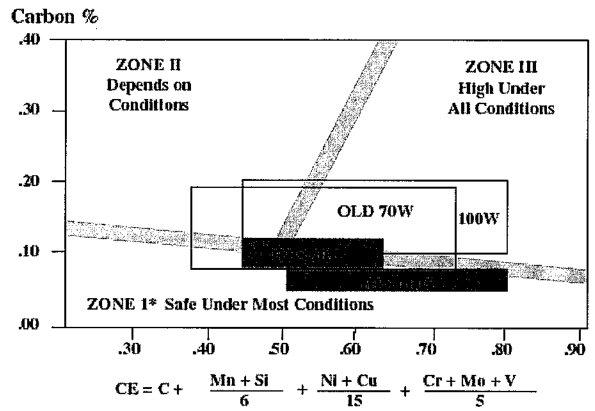


Fig. 2.2.1: The Granville Weldability Diagram

Initial studies on candidate compositions for HPS were evaluated at steel company research laboratories using small laboratory sized heats. These initial studies defined the ability of a candidate steel composition to meet the goal properties. After identifying the chemistries and processing needed for these grades, full-scale mill heats were prepared for the candidate chemistry and full-sized plates produced for evaluation and girder tests. Eventually, a demonstration bridge was identified for initial use of the grade.

2.2.1 HPS 70W

The Grade HPS 70W was the first one developed and commercialized in the HPS program. This grade is now also present in ASTM A709-01a [2.3] and AASHTO M270-02. The chemistry is shown in Table 2.2.2 compared to the Grade 70W that it replaced. HPS 70W is produced by quenching and tempering (Q&T) or Thermo-Mechanical Controlled Processing (TMCP). Grade HPS 70W is available from a number of steel producers. Fig. 2.2.2 presents the distribution of Charpy-V-Notch (CVN) results for over 700 plates, showing the excellent performance of this grade, well above specification minimums. More detailed presentations of Q&T HPS 70W properties have been presented previously, Wilson [2.4]. Because Q&T processing limits plate lengths to 50 ft. (15.2 m) in the U.S., TMCP practices have been developed on the identical HPS 70W chemistry to produce longer plates to 2" (50 mm) thick. The differences in the Q&T and TMCP processing are shown schematically in Fig. 2.2.3. Currently, HPS 70W plates produced by TMCP are available to 1500" (38 m) long depending on weight. Q&T plate girders over 50 ft. (15.2 m) can be specified, but will require additional welded shop splices.
The improved weldability of HPS 70W has been demonstrated to allow welding with limited preheat requirements to 2-1/2 in. (64 mm) thick, whereas the former grade 70W required preheat at thickness over 0.75 in. (19 mm). To take advantage of this improved weldability, new welding consumables were developed for submerged arc welding with low hydrogen conditions, James [2.5]. The fabrication practices for HPS 70W were summarized in a document now available from American Association of State Highway and Transportation Officials (AASHTO) [2.6].

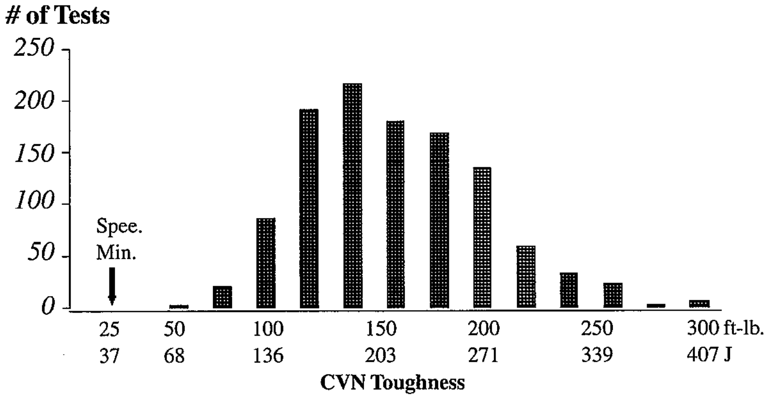


Fig. 2.2.2: Distribution of CVN results for Q&T HPS 70W Plates

2.2.2 HPS 50W

With the successful introduction of HPS 70W, a number of bridge owners requested an HPS version for the more common 50W grade. They were interested in improved weldability and toughness. Steel companies established that an HPS 50W could be provided from the exact chemistry of HPS 70W when processed using conventional hot rolling or controlled rolling (see Fig. 2.2.3). Initial production of HPS 50W has shown excellent CVN toughness levels, easily meeting the specification requirements.

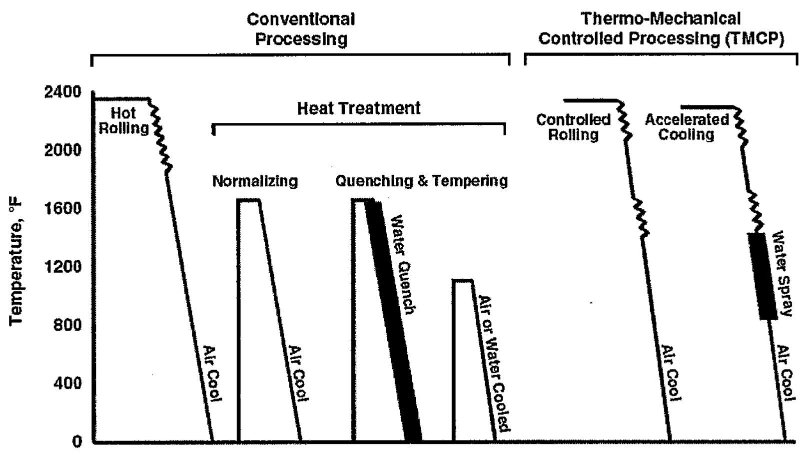


Fig. 2.2.3: Processes for producing plate steels

2.2.3 HPS 100W

The higher strength HPS 100W was a part of the original research program. Currently, a Cu-Ni based grade developed with Lehigh University has been produced as plates from ¼" to 2-1/2" (6.4 – 64 mm) and seamless tubes, Wilson, et al [2.7]. The chemistry specification for this new grade is shown in Table 2.2.2. Initial applications for this grade are in development.

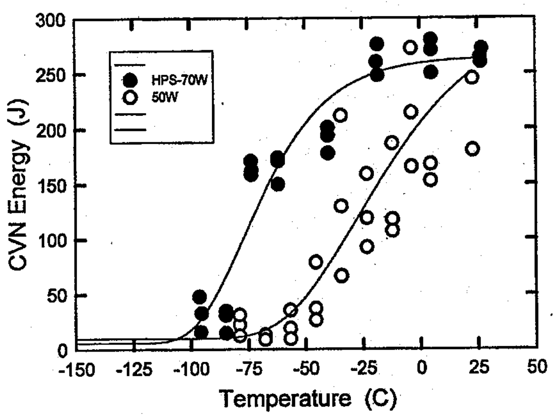
		C	Mn	P	S	Si	Cu	Ni	Cr	Mo	V
Old 70W	Min.	–	0.80	–	–	0.25	0.20	–	0.40	–	0.02
	Max.	0.19	1.35	0.035	0.04	0.65	0.40	0.50	0.70	–	0.10
HPS 70W and HPS 50W (a)	Min.	–	1.10	–	–	0.30	0.25	0.25	0.45	0.02	0.04
	Max.	0.11	1.35	0.020	0.006	0.50	0.40	0.40	0.70	0.08	0.08
ASTM A 1010 (Duracorr®) (b)	Min.	–	–	–	–	–	–	–	10.5	–	–
	Max.	0.03	1.50	0.04	0.030 (C)	1.00	–	1.50	12.5	–	–
Traditional 100W (e, f)	Min.	0.10	0.60	–	–	0.15	0.15	0.70	0.40	0.40	0.03
	Max.	0.20	1.00	0.035	0.035	0.35	0.50	1.00	0.65	0.60	0.08
HPS 100W, Cu-Ni (d, e, g)	Min.	–	0.95	–	–	0.15	0.90	0.65	0.40	0.40	0.04
	Max.	0.08	1.50	0.015	0.006	0.35	1.20	0.90	0.65	0.65	0.08

(a) Calcium treated for inclusion shape control, also requires 0.010–0.040 AI and 0.015 max. N
(b) 0.030 max. N
(c) 0.005 max. for bridge applications
(d) Calcium treated for inclusion shape control
(e) 2–1/2" (65 mm) max. thickness
(f) Contains 0.001B
(g) Contains 0.01/0.03 Nb, 0.02/0.05 AI and 0.015 max. N

Table 2.2.2: Chemistries for conventional and high-performance steels

2.2.4 Fatigue and Fracture Properties

The fatigue resistance of high-performance steels is controlled by the welded details of the connections and the stress range, as is the case for conventional steels. The fatigue resistance is not affected by the type and strength of steels. Tests on high-performance steel conclude that the fatigue categories given in the AASHTO LRFD, Section 6.6.1 Fatigue also apply to high-performance steel welded details. The fracture toughness of high-performance steels is much higher than the conventional bridge steels. This is evident from Fig. 2.2.4, which shows the Charpy-V-Notch (CVN) transition curves for HPS 70W and conventional AASHTO M270 Grade 50W steel. The brittle-ductile transition of HPS occurs at a much lower temperature than conventional Grade 50W steel. This means that HPS 70W remains fully ductile at lower temperatures where conventional Grade 50W steel begins to show brittle behavior.



1 ft.-1 lb. = 0.729 J
1°F = 1.8°C + 32

Fig. 2.2.4: CVN transition curve [2.8]

The current AASHTO CVN toughness requirements are specified to avoid brittle failure in steel bridges above the lowest anticipated service temperature. The service temperatures are divided into three zones as shown in Table 2.2.3.

Minimum Service Temperature	Temperature Zone
0°F and above (-17°C and above)	1
-1° to -30° F (-18°C to -34°C)	2
-31°F to -60°F (-35°C to -51°C)	3

Table 2.2.3: Temperature Zones for CVN Requirements

The AASHTO CVN requirements for these zones are shown in Table 6.6.2-2 Fracture Toughness Requirements in the AASHTO LRFD. The HPS 70W steels tested so far show ductile behavior at the extreme service temperature of -60°F (-51°C) for Zone 3. It is a major accomplishment of the HPS research and an important advantage of HPS in controlling brittle fracture.

With higher fracture toughness, high-performance steels have much higher crack tolerance than conventional grade steels. Full-scale fatigue and fracture tests of I-girders fabricated of HPS 70W in the laboratory showed that the girders were able to resist the full design overload with fracture even when the crack was large enough to cause 50% of loss in net section of the tension flange [2.8]. Large crack tolerance increases the time for detecting and repairing fatigue cracks before the bridge becomes unsafe.

2.2.5 Weldability

A main thrust of the HPS Research Program is to develop bridge steels with significantly improved weldability [2.9]. It is necessary to improve weldability in order to reduce the high cost of fabrication associated with high preheat and interpass temperatures, high heat input, post-weld treatment, and other stringent controls, and to eliminate hydrogen induced cracking in the weldment.

Hydrogen induced cracking, also known as delayed cracking or cold cracking, has been one of the most common and serious problems encountered in steel weldments in bridges. The common source of hydrogen is from moisture. Grease, oxides and other contaminants are also potential sources of hydrogen. Hydrogen from these sources can be introduced into the weld region through the welding electrode, shielding materials, base metal surface and the atmosphere.

Hydrogen-induced cracking can occur in the weld heat affected zone (HAZ) and in the fusion zone (FZ). While the reasons for cracking are the same, controlling the factors that cause cracking can be different for the HAZ and FZ. For the HAZ, control of cracking comes from the modern steel making processes, which incorporate means to avoid susceptible microstructures and eliminate sources of hydrogen in the base metal (steel). For the FZ, control of susceptibility to hydrogen-induced cracking is achieved by adding alloying elements in the consumables and using proper welding techniques, including preheat and heat input.

The most common and effective method of eliminating hydrogen-induced cracking is specifying minimum preheat and interpass temperature for welding. In general, the higher the preheat the less chance for formation of brittle microstructures and more time for the hydrogen to diffuse from the weld. However, preheating is time consuming and costly. One of the goals in developing high-performance steels is to reduce or eliminate preheat. This goal has been successfully accomplished as shown in Table 2.2.4.

	Diffusible Hydrogen		
	H4*	H8*	H16*
AASHTO M270 Grade 70W	212°F (100°C)	248°F (120°C)	248°F (120°C)
HPS 70W	70°F (21°C)	100°F (38°C)	150°F (66°C)

* Denotes the level of hydrogen measured in the laboratory in terms of milliliter per 100 grams of deposited weld metal, e.g. H4 means 4 ml/100g of diffusible hydrogen in the weld metal.

Table 2.2.4: Minimum Preheat

2.2.5.1 AASHTO HPS Guide and AWS Code

The AASHTO HPS Guide and AWS Code contain supplementary welding provisions applicable to HPS. The designers should make sure that the applicable provisions of these two documents are made a part of the contract documents. Some key elements pertaining to welding of HPS are:

- The use of only low-hydrogen practice.
- Only submerged arc (SAW) and shield metal arc (SMAW) welding processes are recommended for HPS at the present. Research is ongoing for the use of gas metal arc (GMAW) welding process.

- The diffusible hydrogen level is limited to a maximum of 8 mL/100g (H8). SAW consumables should be handled such that the diffusible hydrogen is controlled to a level of H4 maximum. SMAW consumables may meet level H4 or H8.
- Consumables with matching weld strength are recommended for SAW complete penetration groove welds connecting Grade HPS 70W plates. Consumables with undermatched weld strength are strongly recommended for all fillet welding. The designers should specify on the contract drawings or special provisions where undermatched fillet welding is permitted or required.
- For connecting HPS 70W to Grade 50W, consumables satisfactory for Grade 50W base metals are considered "matching" strength. However, it is recommended that the diffusible hydrogen level be limited to H4 or H8.

2.2.5.2 Lessons Learned

- SAW consumable combination of Lincoln LA85 electrode and MIL800HPNi flux consistently produces acceptable quality weld metal. This applies to both Q&T and TMCP products.
- It is beneficial to require the fabricator perform weld procedure qualification tests on full size HPS butt welds and HPS to Grade 50W butt and fillet welds using consumable combination proposed for the production welds.
- Improved weldability still needs care and good workmanship to produce quality welds.
- The cost-effectiveness of HPS has been demonstrated by the design and construction of HPS bridges in many states in the U.S.

2.2.6 Weathering Characteristic

It was part of the initial research objective to develop HPS with "weathering characteristic," meaning HPS should have the ability to perform without painting under normal atmospheric conditions. Predictive equations are available in ASTM G101 [2.10] to establish the minimum alloy content required in order to be defined as a weathering steel. An improved set of predictive equations has been developed by Townsend [2.11], which is more useful for higher levels of the key alloy elements. HPS steels have better atmospheric corrosion resistance than the conventional grade 50W or 70W steels.

Recent developments in Japan have reported improved "coastal" weathering steels [2.12]. These grades have higher alloy levels than traditional weathering steels and are reported to perform well in modest chloride environments. Because of the higher alloy levels, the older G101 predictive equation cannot be used. However, the Townsend formulae can, and are now available as an alternative in G101. These Japanese improved weathering steels are compared to HPS 70W and HPS 100W in Fig. 2.2.5. Both HPS grades are predicted to have improved performance in atmospheric corrosion. A new 50 ksi (345 MPa) yield strength, structural stainless steel is being used on bridges. ASTM A1010 (ISG Plate Duracorr) has been specified for two bridge applications (California and Illinois) because of its superior corrosion performance, even in chloride containing environments. The chemistry is shown in Table 2.2.2. Fig. 2.2.5 demonstrates the excellent weathering performance for A1010 with some improvement shown by HPS 70W and the Japanese coastal steels in long-term exposure trials [2.13].

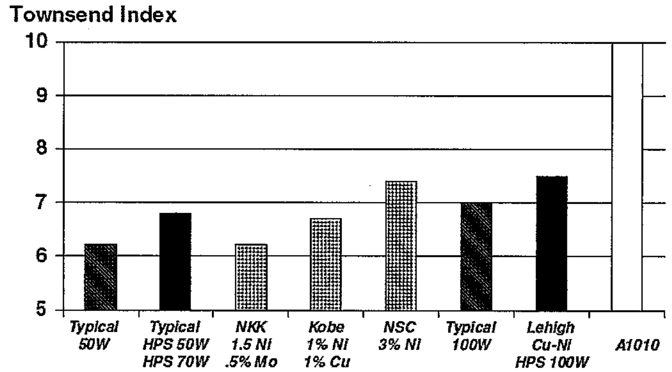


Fig. 2.2.5: Comparison of weathering characteristics of steels

The designers should follow the same guidelines and detailing practice for conventional weathering grade steels to assure successful applications of HPS steels in the unpainted conditions. Guidelines for proper application of unpainted weathering steels in highway bridges may be found in the FHWA Technical Advisory T 5140.22, Uncoated Weathering Steel in Structures, dated October 3, 1989.

2.3 Design and Construction Specifications

The following four documents cover the design, fabrication and construction of steel bridges using high-performance steels:

1 AASHTO LRFD Bridge Design Specifications with Interims (AASHTO LRFD).
2 AASHTO Standard Specifications for Highway Bridges, 17th. Edition, 2002 (AASHTO LFD)
3 AASHTO Guide for Highway Bridge Fabrication with HPS 70W Steel (AASHTO HPS Guide).
4 ANSI/AASHTO/AWS D1.5-95 Bridge Welding Code with Addendums (AWS Code).

These documents reflect the findings and experiences on the applications of HPS by researchers, fabricators, manufacturers, owners and engineers working with high-performance steels.

High-performance steels give the designers another option to achieve durable and cost effective steel bridges [2.14]. HPS design follows the same design criteria and good practice as provided in Section 6 Steel Structures of the AASHTO LRFD Bridge Design Specifications.

Use of HPS 70W generally results in smaller members and lighter structures. The designers should pay attention to deformations, global buckling of members, and local buckling of components. The Service Limit State should be checked for deflection, handling, shipping and construction procedures and sequences.

The live load deflection criteria is considered optional as stated in Section 2, Article 2.5.2.6.2 of the AASHTO LRFD. The reason for this is that past experience with bridges designed under the previous editions of the AASHTO Standard Specifications has not shown any need to compute and control live load deflections based on the heavier live load required by AASHTO LRFD. However, if the designers choose to invoke the optional live load deflection criteria specified in Article 2.5.2.6.2, the live load deflection should be computed as provided in Section 3, Article 3.6.1.3.2 of the AASHTO LRFD. It may be expected that designs using all HPS 70W or HPS 100W would exceed the deflection limit of L/800. In such a case, the designers have the discretion to exceed this limit or to adjust the sections by optimizing the web depth and/or increasing the bottom flange thickness in the positive moment region or use a hybrid design to keep the deflection within limit. The AASHTO HPS Guide encourages the use of hybrid girders, i.e. combining the use of HPS 70W and Grade 50W steels. A hybrid combination of HPS 70W in the negative moment regions and Grade 50W or HPS 50W in other areas results in the optimum use of HPS and attains the most economy.

2.4 HPS Design Experience

Many State Departments of Transportation (DOT) have designed and constructed HPS bridges. Several organizations have done comparative designs to optimize the use of HPS in combination with other grades of steels. Brief descriptions of the design experience and cost studies of some of the State DOTs and organizations are given in the following sub-sections.

2.4.1 First HPS 70W Bridge

Nebraska DOT was the first to use HPS 70W in the design and construction of the Snyder Bridge – a welded plate girder steel bridge as shown in Fig. 2.4.1.
The bridge was opened to traffic in October 1997. It is a 150-foot (45.7 m) simple span bridge with 5 lines of plate girders of 4' 6" (1.37 m) deep. The original design utilized conventional grade 50W steel. When HPS first became available, Nebraska DOT replaced the grade 50W steel with HPS 70W steel of equal size. The intent was to use this first HPS 70W bridge to gain experience on the HPS fabrication process. The fabricators concluded that there were no significant changes needed in the HPS fabrication process.

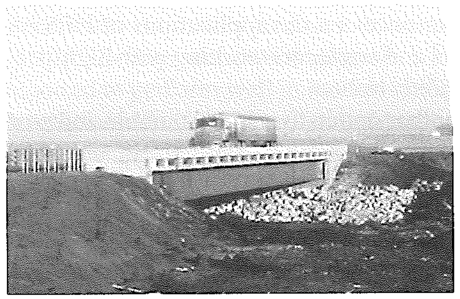


Fig. 2.4.1: Snyder Bridge

2.4.2 The Nebraska HPS Two-Box Girder System [2.15]

The Nebraska DOT in cooperation with the National Bridge and Research Organization commissioned J. Muller International to develop an innovative concept optimizing the use of HPS. The result of this initiative is a two-box girder bridge with full depth composite deck system. The cross section of the system is shown in Fig. 2.4.2. This system has two spans of 120 feet (36.6 m) each. It is designed for two lanes of traffic with wide shoulder, measuring 44' (13.4 m) curb to curb. The system can be used for new bridges and to replace many existing grade separation structures.

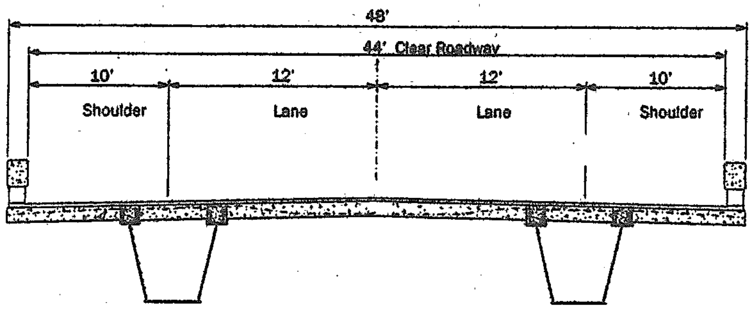


Fig. 2.4.2: Nebraska HPS Twin-Box

A two-box girder system was selected because of its simplicity, small size, efficiency of load distribution, stability for handling and erection, and robustness against vehicle impact. The substructure might be semi-integral or fully integral abutment to eliminate joints and bearings. The superstructure might be semi-continuous or continuous for live load only, or fully continuous for dead load and live load.

2.4.3 HPS Cost Study [2.16]

HDR Engineering, Inc. in association with the University of Nebraska-Lincoln performed a study to compare the cost differences between bridge designs using HPS 70W, conventional grade 50W and a combination of the two grades of steels. A total of 42 different girder designs were made using the AASHTO LRFD Bridge Design Specifications – HL-93 Live Load. The girder designs had 2-span continuous layout, covering a span range of 150', 200' and 250' (47.7, 61.0 and 76.2 m), variable girder spacing of 9' and 12' (2.7 and 3.6 m), and designs in grade 50W, HPS 70W and a variety of hybrid combinations.
The following unit cost data was obtained from the fabricators and used in the relative cost comparison of the various designs:

	Material Cost	Fabricated Cost
Grade 50W	\$0.40/lb. (\$0.88/kg)	\$0.61/lb. (\$1.34/kg)
HPS 70W Q&T	\$0.54/lb. (\$1.19/kg)	\$0.75/lb. (\$1.65/kg)
HPS 70W TMCP	\$0.51/lb. (\$1.12/kg)	–

Table 2.4.1: Cost comparison

The study concludes that:
1 HPS 70W results in weight and depth savings for all span lengths and girder spacing.
2 Hybrid designs are more economical for all of the spans and girder spacing. The most economical hybrid combination is grade 50 for all webs and positive moment top flanges, with HPS 70W for negative moment top flanges and all bottom flanges.
3 LRFD treats deflection as an optional criterion with different live load configurations. If a deflection limit of L/800 is imposed, deflection may control HPS 70W designs for shallow web depth.

2.4.4 Tennessee Experience [2.17]

In 1996, Tennessee Department of Transportation (TNDOT) was completing the design of the SR53 Bridge over the Martin Creek using ASTM A709 Grade 50W steel (See Fig. 2.4.3). It was TNDOT's first steel bridge design utilizing the new AASHTO LRFD Bridge Design Specifications. The bridge consisted of two 235.5-foot (71.8 m) spans, carrying a 28-foot (8.5 m) roadway on three continuous welded plate girders spaced at 10' 6" (3.20 m) on centers.
At about the same time, HPS 70W steel became available. With support from FHWA, TNDOT offered to test the application of HPS 70W in an actual bridge. In order to provide a true comparison, TNDOT optimized the redesign using HPS 70W for the girders and Grade 50W shapes for the cross-frames. The HPS redesign resulted in 24.2% reduction in steel weight and 10.6% savings in cost. The weight and cost savings of the HPS 70W bridge are shown below.

	Conventional Grade 50W	HPS 70W & Grade 50W
Steel Weight	675,319 lb. (306 t)	511,908 lb. (232 t)
In-Place Cost	$1.00/lb. ($2.20/kg)*	$1.18/lb. ($2.60/kg)**
Total Steel Cost	$675,319	$604,051

* Construction cost in Tennessee in 1996
** This unit cost includes change orders for $25,000 for additional shop splices and $10,000 for change in welding flux.

Table 2.4.2: Cost comparison

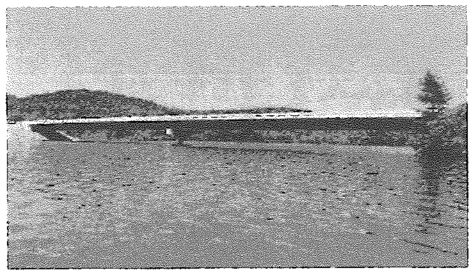


Fig. 2.4.3: Martin Creek Bridge

The bridge was opened to traffic in February 1998. Since then, TNDOT has completed two more HPS 70W bridges. The TNDOT is satisfied with the three HPS 70W bridges constructed to date. TNDOT is currently designing more HPS bridges utilizing HPS 70W TMCP for webs and flanges, and Grade 50W in other members. The use of HPS has become a routine practice in Tennessee.
Some of Tennessee's optimization techniques are:
- Use uncoated HPS steels.
- Use HPS 70W steel for flanges and webs over interior supports, where moments and shears are high.
- Use hybrid girder sections for composite sections in positive bending, where moments are high, but shears are low.
- Use undermatching fillet welds with HPS 70W to reduce cost of consumables.
- Use constant width plates to the greatest extent possible. Consider plate width changes at field splices wherever practical.
- Consider waiving live load deflection limits for lane loads.
- Use TMCP plates to the greatest extent possible.
- Use the new AASHTO Guide for highway bridge fabrication with HPS 70W Steel. Recommendations in the Guide should be followed, with no more stringent requirements added.

2.4.5 Pennsylvania Experience [2.18]

The Pennsylvania Department of Transportation (PennDOT) has used HPS 70W in the Ford City Bridge (See Fig. 2.4.4), which was opened to traffic in July 2000. Penn DOT performed full-scale tension and fatigue testing, extensive material testing and weld testing in this project.
It is a three-span continuous welded steel plate girder bridge with spans of 320'-416'-320' (98-127-98 m). The first span is curved horizontally with a radius of 508' (158 m). The other two spans are on tangent. There are four lines of girders spaced at 13.5' (4.1 m). HPS 70W is used in the negative moment regions and grade 50W elsewhere. This hybrid combination of steels resulted in 20% reduction in steel weight, and enabled the girder sections to be constant depth instead of haunched. By eliminating the variable web depth, a costly longitudinal bolted web splice was avoided.

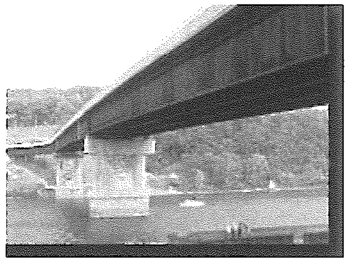


Fig. 2.4.4: Ford City Bridge

PennDOT has several HPS bridges under construction and design. PennDOT is sponsoring research to realize additional benefits from HPS. It intends to construct an HPS demonstration bridge with innovative corrugated web I-girder. PennDOT is optimistic that HPS will reduce bridge construction costs.

2.4.6 New York State Thruway Authority Experience [2.19]

The New York State Thruway Authority (NYSTA) participated in a cooperative agreement with the Federal Highway Administration (FHWA) to evaluate and document the use of HPS 70W on bridges fabricated and constructed at various locations on the limited access highway system. Under this agreement, NYSTA constructed 7 structures.

The first project was the Berkshire Thruway over the Muitzes Kill Bridge (Fig. 2.4.5) using HPS 70W. It is a 200 ft. (61 m) long simple-span, jointless bridge carrying two lanes of traffic and consisting of six 72 in. (1.8 m) deep plate girders spaced at 8 ft. (2.4 m) centers. This bridge was originally designed as a two-span structure using conventional Grade 50W steel. The plans were revised to take advantage of the strength of HPS 70W by eliminating an interior pier.

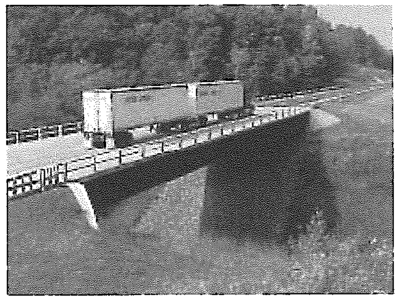


Fig. 2.4.5: Berkshire over Muitzes Kill Bridge

The second project was the I-90 Exit 54 Interchange Overpass (Fig. 2.4.6). It is a two-span, jointless bridge consisting of nine 29 in. (0.74 m) deep plate girders spaced at 7.5 ft. (2.29 m) centers. The span lengths are 105.5 ft. and 98.5 ft. (32.2 and 30.0 m). The primary stress-carrying members, including stiffeners and connection plates, were fabricated of HPS steel. The secondary members were fabricated of Grade 50W steel.

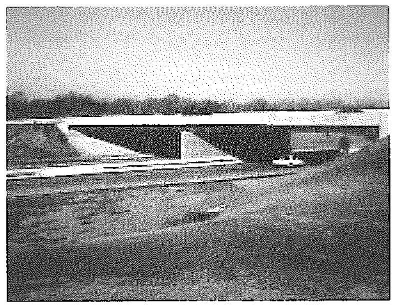


Fig. 2.4.6: I-90 Exit 54 Interchange Overpass

Next, a series of five bridges that carry local traffic over I-90 were constructed of HPS 70W steel. The NYSTA engineers took advantage of the high strength of HPS 70W to design two-span jointless structures to replace the existing deficient four-span structures. A typical two-span structure is similar to that shown in Fig. 2.4.6, and consists

of 5 lines of 29 in. (0.74m) deep plate girders spaced at 7.5 ft. (2.29 m) centers. The span lengths are 100 ft. (30.5 m) each. For all these bridges, weld procedure qualification tests and diffusible hydrogen tests were conducted prior to commencing fabrication. These tests used the submerged arc welding (SAW) process with matching consumables, i.e. Lincoln LA-100 electrodes in combination with Lincoln Mil800H flux. These bridges were welded in accordance with the AASHTO Guide for Highway Bridge Fabrication with HPS 70W Steel.

2.5 HPS Fabrication Experience

Fabricators of HPS have reported varied experiences with drilling HPS 70W Q&T steel. The experiences range from "no difference than Grade 50W steel" to "drills and reamers dull quickly". The HPS Steering Committee recommends that drilled or reamed holes be flooded with lubricant during drilling or reaming.

Fabricators also note that mill scale removal by abrasive blasting requires about the same work effort as Grade 50W steel. However, mill scale removal by descaler or grinding is very difficult for the HPS Q&T steel. Fabricators report that there is no difference in flame cutting procedures when compared with Grades 36, 50 or 50W steels. Flame cut edges of HPS steels do not get excessively hardened (RC30 or higher) as in the case of flame cut edges of grade 50W steel. Predicting pre-cut camber gain or loss requires experimentation, especially with Q&T steels.

Welding of HPS 70W steel is currently restricted to the submerged arc and shielded metal arc welding processes for both Q&T and TMCP products. Other arc welding processes are being studied. It is expected that other welding processes commonly used in bridge fabrication will be approved in due course.

2.6 Availability and Cost

2.6.1 Availability

Currently, HPS 50W, HPS 70W and HPS 100W steels are available in plates in thicknesses shown in Table 2.2.1. Steel producers will welcome inquiry for custom orders of special sizes. The delivery time is approximately 6 to 10 weeks depending on market demand.

Based on industry information, the steel industry has responded to a market demand of over 450,000 tons of steel bridge fabrication in 2003. Steel producers are well positioned to meet the growing demands for HPS in the years ahead.

2.6.2 Cost

The approximate unit prices for materials, fabricated members and in-place cost of steel structures are shown below:

	Material	Fabricated	In-Place
Grade 50W	$0.35–0.42/lb. ($0.77–0.93/kg)	$0.55–0.62/lb. ($1.21–1.37/kg)	$1.00–1.25/lb. ($2.20–2.76/kg)
HPS 50W	$0.42–0.50/lb. ($0.93–1.10/kg)	$0.63–0.71/lb. ($1.39–1.57/kg)	$1.15–1.40/lb. ($2.54–3.09/kg)
HPS 70W Q&T	$0.48–0.60/lb. ($1.06–1.32/kg)	$0.75–0.83/lb. ($1.65–1.83/kg)	$1.18–1.50/lb. ($2.60–3.31/kg)
HPS 70W TMCP	$0.45–0.57/lb. ($0.99–1.27/kg)	$0.70–0.78/lb. ($1.54–1.72/kg)	$1.15–1.45/lb. ($2.54–3.20/kg)

Table 2.6.1: Cost comparison

The actual unit cost for a project is expected to vary from region to region, from structure to structure depending on complexity, and to change based on market conditions. It is important that the designers establish open communication with the local suppliers and the fabricators of HPS steels. The communication should start from preliminary design to final design and plan, specification and estimate (PS&E). This open communication between parties with specific knowledge and experience will lead to cost effective design and successful construction. The latest information on availability and cost can best be obtained from local suppliers and fabricators when the PS&E is almost complete and the Engineer's Cost Estimate is being prepared.

2.7 Producers and Industry Organizations

The major U.S. manufacturers of HPS steels are:
- ISG Plate, Inc.
  139 Modena Road
  P.O. Box 3001
  Coatesville, PA 19320
  Phone: 610-383-3105
  www.intlsteel.com

- Oregon Steel Mills, Inc.
  14400 N Rivergate Boulevard
  Portland, OR 97203
  Phone: 503-240-5240
  www.oregonsteel.com

The following industry organizations can provide technical information regarding the design, specifications and fabrication of steel bridges using high-performance steels:
- American Institute of Steel and Iron (AISI) website:
  1140 Connecticut Avenue, Suite 705
  Washington, DC 20036
  Phone: 202-452-7100
  www.steel.org

- AISC/National Steel Bridge Alliance (NSBA) website:
  One East Wacker Drive, Suite 3100
  Chicago, IL 60601
  Phone: 312-670-2400
  www.aisc.org

These plate producers and industry organizations can best provide the latest project specific information on availability, delivery schedule and cost.

2.8 Websites

The following websites contain a wealth of information on research, design, fabrication, construction, committee and advisory group activities, conferences and other technical aspects of HPS. They also link to other websites for additional information.

- The American Iron and Steel Institute (AISI)
  Website: http://www.steel.org/infrastructure/bridges
- FHWA, Office of Bridge Technology
  Website: http://www.fhwa.dot.gov/bridge
- National Bridge Research Organization (NaBRO),
  University of Nebraska-Lincoln
  Website: http://www.nabro.unl.edu

2.9 Continuing Research

The transition from research to practice has been very swift for high-performance steels. In less than 3 years after the initiation of the joint research effort by AISI, FHWA and the Navy, HPS 70W steel plates became commercially available and used successfully by Nebraska and Tennessee for bridge design and construction. Many states are now using HPS steels. With continuing funding from the steel industry, state and federal governments, the HPS Research Program will continue to make improvements in material, design, fabrication and construction.
The following research is underway in four disciplines:
1 Steel: initial applications of HPS 100W; add HPS 100W and A1010 to ASTM and AASHTO specifications when appropriate.
2 Welding: evaluate the use of different welding methods – gas metal arc welding (GMAW) and flux cored arc welding (FCAW); develop optimized welding consumables for HPS 100W; update AWS D1.5 Welding Code; update HPS 70W Fabrication Guide.
3 Corrosion: use accelerated and long-term test methods to compare improved weathering HPS steels; assist FHWA in updating Technical Advisory on Weathering Steel.
4 Design: evaluate design constraints for high strength steels; take full advantage of the high toughness of HPS; develop innovative designs.

2.10 Closing Remarks

The development of high-performance steels is a very successful story of putting research into practice in a very short span of time. It exemplifies the vision and leadership of a strong and collaborative partnership between governmental agencies, industry and academia. In recognition of the accomplishments of the joint effort, the Civil Engineering Research Foundation (CERF) awarded the Charles Pankow Award to AISI, FHWA and the Navy.
The cost effective application of HPS in bridge design and construction has already been demonstrated by the successful performance experience of in-service HPS bridges in over 40 states. The population of HPS bridges is growing rapidly in the United States as shown in Fig. 2.10.1.

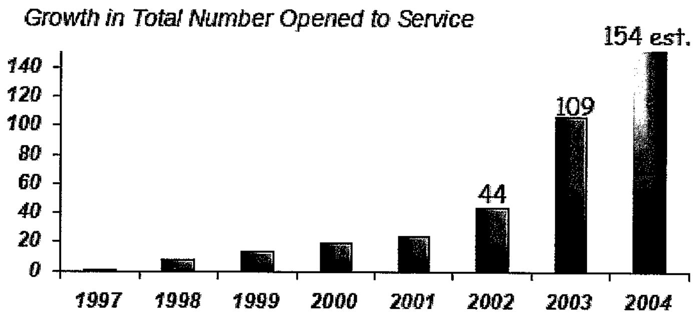


Fig. 2.10.1: HPS Bridges in Service in the U.S.

The HPS program has resulted in the development of new steel grades HPS 50W, HPS 70W, and HPS 100W with improved weldability, toughness and weathering performance. Improved welding consumables and practices have also been developed for these steels.
Significant progress is being made in the AASHTO design codes to more efficiently use high strength steels in bridges. The HPS research effort has continuing challenges in order to develop more cost effective steel bridge designs.
The major benefits of HPS are noted below:
- The high strength of HPS allows the designers to use fewer lines of girders to reduce weight and cost, use shallower girders to solve vertical clearance problem, and increase span lengths to reduce the number of piers on land or obstructions in the streams.
- Improved weldability of HPS eliminates hydrogen induced cracking, reduces the cost of fabrication by lower preheat requirement, and improves the quality of weldment.
- Significantly higher fracture toughness of HPS minimizes brittle and sudden failures of steel bridges in extreme low service temperatures. Higher fracture toughness also means higher cracking tolerance, allowing more time for detecting and repairing cracks before the bridge becomes unsafe.
- Good "weathering characteristic" of HPS assures long-term performance of unpainted bridges under atmospheric conditions.

- Optimized HPS girders can be attained by using a hybrid combination of HPS 70W in the negative moment top and bottom flanges, and Grade 50W or HPS 50W in other regions.
- Optimized HPS girders have been shown to result in lower first cost and are expected to have lower life-cycle cost.

High-performance steels are justifiably claimed to be "The Bridge Construction Material for the New Century."

2.11 References

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2.12 Acknowledgements

The research work reported herein was carried out under the guidance of the FHWA/AISI/Navy High-Performance Steel Steering Committee. Some of the research work was reported to the Carderock Division, Navy Surface Warfare Center and was sponsored by the Federal Highway Administration. The commercial plate production was carried out at International Steel Group, Inc. (ISG), formerly Bethlehem Steel Corporation, facilities in Burns Harbor, IN, Coatesville, PA and Conshohocken, PA.
The authors draw upon the fine works and experiences of many colleagues and professionals from academia, industry and government. They wish to express their appreciation to these colleagues and professionals who so generously share information through personal contact, research reports, conference proceedings, case studies, and presentations at technical committee meetings.
The authors welcome comments, design tips, efficient structural details, ideas and suggestions from the readers for continuous improvement in the application of HPS in durable and cost effective bridge design and construction.

2.13 Appendices

APPENDIX A – SAMPLE HPS SPECIAL PROVISIONS (From AISI website)
Special Provisions: High-Performance Steel, Grade HPS 70W, For Bridge Applications

Table of Contents:

1. DESCRIPTION:
1.01. Under this work, the Contractor shall fabricate, furnish and erect structural steel in accordance with the Contract documents.

1.02. This specification applies to the fabrication of structural components for bridges using quenched and tempered (Q&T) high-performance steel plates, non quenched and tempered, thermo-mechanical-controlled-processing (TMCP) high-performance steel plates, or hybrid/mixed design structural components using high-performance steel plates (Q&T and/or TMCP) in combination with high strength, low alloy steel plates and shapes, for welded or bolted applications in bridge construction.

1.03. All provisions of Section ___ of the ______________ State Standard Specifications, shall apply, except as modified on the plans or in this specification.

2. MATERIALS:
2.01. All steel must comply with all provisions of the latest edition of ASTM A709/A709M, Standard Specification for Carbon and Low Alloy Structural Steel Shapes, Plates and Bars, and Quenched and Tempered Alloy Structural Steel Plates for Bridges, except as modified herein. Supplementary Requirement S83, Non-Fracture Critical Materials Toughness Tests and marking, or S84, Fracture Critical Materials Toughness Tests and marking will apply, as appropriate, and must be specified with the mill order.

2.02. All quenched and tempered high-performance structural steel must conform to the provisions the latest edition of ASTM A709, Grade HPS 70W with a minimum specified yield point of 70 ksi/485 MPa, except as modified below.

2.03. The Contractor is advised that quenched and tempered ASTM A709, Grade HPS 70W steel plates are limited to a 50 feet/15.24 m maximum delivery length from the mill.

2.04. As an option, HPS 70W thermo-mechanical-controlled-processing (TMCP) steel plates with a minimum specified yield point of 70 ksi/485 MPa are also available from the manufacturer in limited thicknesses, and may be directly substituted for the quenched and tempered product. The HPS 70W TMCP product is not currently included in ASTM A709/A709M specifications. However, except for the rolling and heat treating processes, the manufacture, testing, delivery and requirements for mill inspection of HPS 70W TMCP steel must comply with all provisions of the latest edition of ASTM A709/A709M, Standard Specification for Carbon and Low Alloy Structural Steel Shapes, Plates and Bars, and Quenched and Tempered Alloy Structural Steel Plates for Bridges. Supplementary Requirement S83, Non-Fracture Critical Materials Toughness Tests or S84, Fracture Critical Materials Toughness Tests and marking will apply, as appropriate, and must be specified with the mill order.

2.05. Fabrication:
2.05.A. All fabrication must conform to the latest edition of the AASHTO Guide Specifications for Highway Bridge Fabrication with HPS 70W Steel, an addendum to ANSI/AASHTO/AWS D1.5-95, except as modified herein.

2.05.B. Restrictions:

2.05.B.1. Short term application of heat for purposes of heat curving, heat straightening, camber and sweep adjustment, or other reasons, is limited to 1100 F/590 C maximum. All applications of heating must be done by procedures approved by the Chief Engineer or his authorized representative.

2.06. Welding:
2.06.A. All welding must conform to the latest edition of the AASHTO/ AWS D1.5 Bridge Welding Code, except as modified herein and by the latest edition of the AASHTO Guide Specifications for Highway Bridge Fabrication with HPS 70W Steel, an addendum to ANSI/AASHTO/AWS D1.5-95.

2.06.B. Caution:

2.06.B.1. The matching submerged arc welding consumables ESAB ENi4 electrode in combination with Lincoln Mil800H, recommended in Appendix A of the AASHTO Guide Specifications for Highway Bridge Fabrication with HPS 70W Steel, has produced weldments containing unacceptable discontinuities in a substantial number of complete penetration groove welds in one structure, based on the parameters used and experience of one fabricator. In September 2000, the HPS Steering Committee rescinded its recommendation of this combination of welding consumables. Therefore, the ESAB ENi4 electrode in combination with Lincoln Mil800H flux will not be allowed.

2.06.C. Only submerged arc and shielded metal arc welding processes will be permitted when welding Grade HPS 70W steel. Consumable handling requirements shall be in accordance with AWS D1.5, Sections 12.6.5 and 12.6.6, except that SAW consumables shall meet the hydrogen control level of H4 as discussed in AWS D1.5, Section 12, Article 12.6. SMAW consumables can meet either H4 or H8 except the higher preheat and interpass temperatures as noted in Table 3 of the AASHTO Guide Specifications for Highway Bridge Fabrication with HPS 70W Steel apply to H8 conditions.

2.06.D. Filler metals used to make single pass fillet welds for web to flange applications, and for attaching stiffeners and connection plates to Grade HPS 70W webs and flanges, must be in conformance with AWS D1.5, Table 4.1 for ASTM A709 Grade 50W base metal. Filler metals for single pass fillet welds need not meet the requirements for exposed bare applications.

2.06.E. Filler metals used for all complete penetration groove welds connecting Grade HPS 70W plate to ASTM A709, Grade 50W plate may conform to the requirements for welding Grade 50W base metal, or may conform to the requirements for welding Grade HPS 70W base metal as listed below.

2.06.F. Filler metals used for all complete penetration groove welds connecting Grade HPS 70W plates shall conform to the requirements for HPS 70W base metal as follows:
1. Submerged Arc Welding process:
Wire – LA85 by Lincoln Electric Company
Flux – MIL800HPNi by Lincoln Electric Company
2. Shielded Metal Arc Welding process
Matching – E9018MR*
Undermatching – E7018MR*

* the designator "MR," for moisture resistant coating, is required for all SMAW electrodes used for welding HPS 70W steels.

2.06.G. The Contractor may request approval of alternate consumables in lieu of the above filler metals for SAW. The request for approval must include documentation of successful welding in accordance with the AWS D1.5 Bridge Welding Code, and include diffusible hydrogen tests as described in AWS D1.5, Article 12.6.2 indicating the deposited weld metal under proposed fabrication shop conditions has a diffusible hydrogen level equivalent to H4 or less.

2.06.H. All welding procedures must be qualified in accordance with AWS D1.5, Section 5, Qualification. In general, the provisions of Article 5.12 shall apply. Qualification tests shall measure strength, toughness and ductility, with results evaluated in accordance with Article 5.19. If specified on the plans, additional tests shall measure the Charpy V-notch toughness of the coarse grained area of the heat affected zone (HAZ). The notch in the specimens shall be carefully located in the coarse grained area of the HAZ, as determined by macro-etching the specimens prior to machining and testing. The toughness requirement for the HAZ shall be the same as the weld metal.

2.06.I. All procedure qualification tests must be ultrasonically tested in conformance with the requirements of AWS D1.5-95, Section 6, Part C. Evaluation must be in accordance with AWS D1.5-95, Table 9.1, Ultrasonic Acceptance – Rejection Criteria – Tensile Stress. Indications found at the interface of the backing bar may be disregarded, regardless of the defect rating.

2.06.J. A representative of the owner must witness all welding procedure specification qualification tests.

2.06.K. Results of the welding procedure specification qualification tests and final welding procedure specifications must be submitted to the Chief Engineer or his authorized representative for review and approval.

2.06.L. In general, post weld heat treatment shall not be required. The use of such post weld heat treatment shall require additional qualification testing.

2.06.M. Welders and welding operators must be qualified in accordance with the provisions of the AASHTO/ AWS D1.5 Bridge Welding Code.

3. CONSTRUCTION DETAILS:
3.01. All structural steel work, including but not limited to shop drawings, fabrication, inspection, transportation and erection must be done in accordance with the provisions of the Standard Specifications, except as modified by the Contract documents.

3.02. Only fabricators meeting the requirements of the AISC Quality Certification Program, "Major Steel Bridges (Cbr)" with "Fracture Critical Members Endorsement (F)", or approved equal, may be used to fabricate HPS 70W steel. Prior to approval for fabrication, the results of the latest AISC certification review shall be made available to the owner's representative to determine if items critical to successful fabrication meet the needs of the specific work.

3.03. Whenever magnetic particle testing is done, only the yoke technique will be allowed, as described in Section 6.7.6.2 of the AASHTO/ AWS D1.5 Bridge Welding Code, modified to test using alternating current only. The prod technique will not be allowed.

4. METHOD OF MEASUREMENT:
4.01. No measurement shall be taken.

5. BASIS OF PAYMENT:
5.01. The lump sum price shall include the cost of furnishing all labor, materials, and equipment necessary to complete the work, as described on the plans, in this specification and in the applicable sections of the Standard Specifications.

APPENDIX B – DESIGN ALTERNATIVES FOR COST-EFFECTIVE USE OF HIGH-PERFORMANCE STEEL
In 1996 Federal Highway Administration (FHWA), the American Iron and Steel Institute, and the Office of Naval Research introduced High-Performance Steel (HPS) grades with higher strength, superior toughness, improved weldability and higher corrosion resistance compare to conventional grade 50W steel. Since then, HPS 70W has been used successfully on bridges across the United States and is growing in popularity among bridge engineers. As HPS 70W steel becomes a more viable alternative, designers are faced with two basic questions, when is HPS 70W steel economical and what are its advantages. To answer these questions, FHWA sponsored a study to develop a series of steel girder designs using HPS 70W and conventional grade 50W and compare the performance and costs. The study evaluated a broad range of girder configurations differing in span length, girder spacing, and material compositions. Fabricator input was solicited to develop relative unit costs, and those costs were applied to the various design combinations to obtain realistic comparisons.
The study selected a series of two-span continuous, symmetric bridge configurations to account for the effects of both positive- and negative-moment regions. Two-span grade separation structures are also an effective application of HPS because the longer spans make it possible to eliminate shoulder piers and thus increase highway safety. The study also covered a range of span lengths, which included 100 ft, 150 ft, 200 ft, and 250 ft (30.5, 45.7, 61.0 and 76.2 m).
Each bridge was designed with both a five-girder and a four-girder cross section to evaluate the effect of girder spacing. Girder spacing varied from 9 to 12 ft. (2.7 and 3.7 m) Fig. B.1 shows the cross section used in the study for the four-girder and five-girder options.
Additionally, each of the resulting bridge configurations was designed for different material combinations, among them homogeneous 50W steel, homogeneous HPS 70W steel, and five different hybrid sections. Each girder was also designed for a range of web depths (three to four) to determine the optimum depth for each of the various combinations. The flowchart of Fig. B.2 illustrates all of the combinations of span lengths, and minimum number of web depth ratios considered. The study resulted in 49 girder designs at optimal depth, with a total of nearly 170 girder designs necessary to determine those depths. Fig. B.3 shows the five hybrid combinations of grade 50 and grade 70 steel in the webs and flanges within the positive- and negative-moment regions.

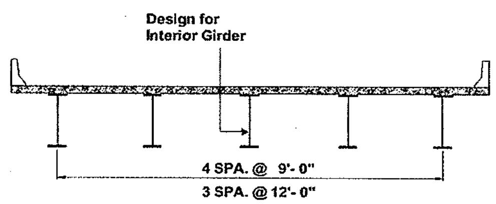


Fig. B.1: Typical Cross Section

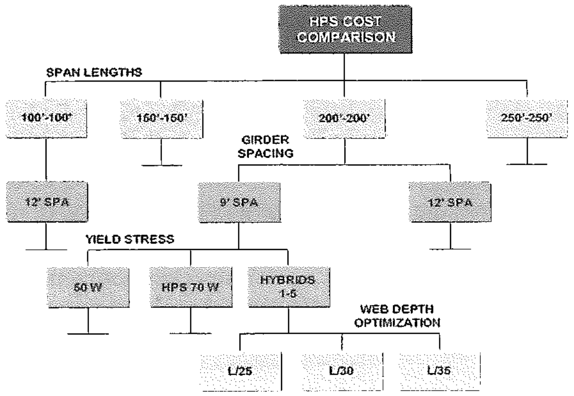
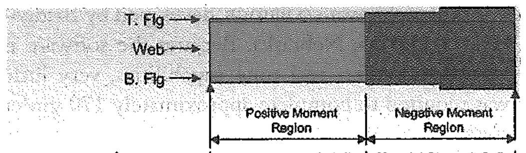


Fig. B.2: Design Flowchart



Hybrid Number		Positive Moment Region: Material Strength (ksi)	Negative Moment Region: Material Strength (ksi)
1	T. Flg	50	70
	Web	50	50
	B.Flg	50	70
2	T. Flg	70	70
	Web	50	50
	B.Flg	70	70
3	T. Flg	70	70
	Web	50	70
	B.Flg	70	70
4	T. Flg	50	70
	Web	50	50
	B.Flg	70	70
5	T. Flg	50	70
	Web	50	70
	B.Flg	70	70

Fig. B.3: Hybrid Design Combinations

The girders were designed in conformity with the second edition of AASHTO LRFD Bridge Design Specifications, published by American Association of State Highway and Transportation Officials (AASHTO).

A partially stiffened design approach was selected for web thickness and stiffeners. It was assumed that the stiffener cost was roughly four times that of the web and flange material, and this assumption was later substantiated by the fabricator cost data. Girder web depth optimization was also performed on all 9 design combinations.

The relative performance considerations between the steel types included items such as weight savings, structure depth reduction, and effect of deflection and fatigue. The performance comparison is based on designs composed entirely of 50W steel and entirely of HPS 70W steel. Although the hybrid sections offered certain performance advantages, the benefits fall somewhere between the bounds set by the 50W and HPS 70W designs.

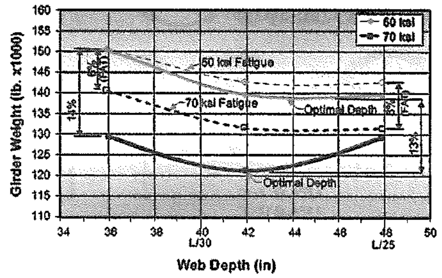


Fig. B.4: Girder weight versus web depth (100 ft. spans and 12 ft. girder spacing)

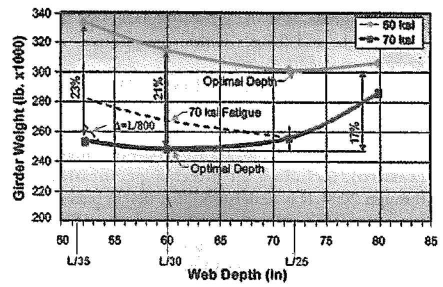


Fig. B.5: Girder weight versus web depth (150 ft. spans and 12 ft. girder spacing)

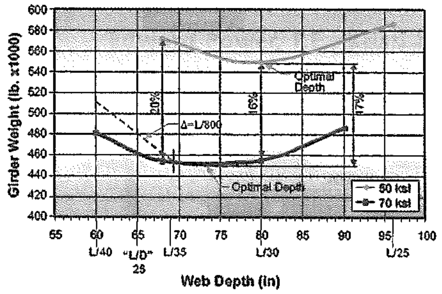


Fig. B.6: Girder weight versus web depth (200 ft. spans and 12 ft. girder spacing)

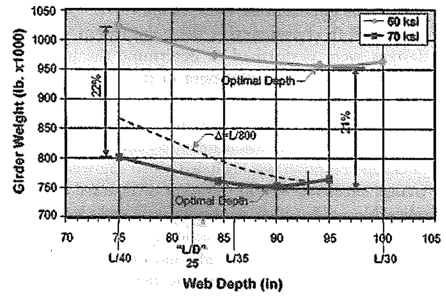


Fig. B.7: Girder weight versus web depth (250 ft. spans and 12 ft. girder spacing)

Fig. B.4 through Fig. B.7 show the girder weight versus web depth graphs for the four span-length configurations and a girder spacing of 12 ft. (3.7 m). The weight plotted is the total of all girders in the cross section. Comparisons of performance for the 9 ft. (2.7 m) girder spacing yield similar results, except that total girder weights are more, as would be expected with an additional line of girders. As seen from the graphs, the optimal girder depth varies between HPS 70W and 50W steels, the former typically being less.

The following Table B.1 summarizes the weight savings for 9 ft and 12 ft (2.7 and 3.7 m) girder spacing conferred by the HPS 70W and hybrid-4 combination designs over 50W design at optimal depth.

Spans	Girder Spacing	HPS 70W (%)	Hybrid-4 (%)
100 ft. – 100 ft. (30.5 m – 30.5 m)	12 ft. (3.7 m)	13 (8)	13 (8)
150 ft. – 150 ft. (45.7 m – 45.7 m)	9 ft. (2.7 m) 12 ft. (3.7 m)	17 (13) 17 (15)	11 (9) 16 (14)
200 ft. – 200 ft. (61.0 m – 61.0 m)	9 ft. (2.7 m) 12 ft. (3.7 m)	17 17	16 15
250 ft. – 250 ft. (76.2 m – 76.2 m)	9 ft. (2.7 m) 12 ft. (3.7 m)	17 12	13 20

*Values in parentheses represent weight savings with fatigue taken into consideration.

*Table B.1: Weight Savings at Optimal Depth compared to Grade 50W**

The graphs also illustrate the increased weight savings trend at web depths less than optimum, which is important for design requiring shallow girder depths. Savings above the optimum depth becomes less; however, sections in this range are rarely used because deeper girders typically do not offer any advantages. Based on the designs encompassed in this study, the use of HPS 70W will result in weight savings of 13 to 25% for practical girder depths.

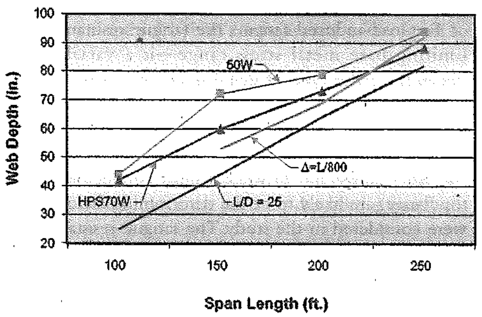


Fig. B.8: Optimal depth and deflection

The load and resistance factor design (LRFD) specifications differ from AASHTO's standard specifications in that they treat deflection as an optional criterion with a slightly different loading configuration. The designs performed for these studies were sized on the bases of bending and shear but were not limited by deflection. Considering a deflection limitation of L/800, the designs were reviewed for compliance with that limitation. The dashed line in Fig. B.5 through Fig. B.7 shows the web depth range where the original designs do not meet L/800 deflection limit. It also shows the additional girder weight required to meet the optional deflection limit. In all configurations, the designs using 50W steel did not exceed this deflection limit. The HPS 70W steel designs exceeded the allowable deflection for some web depths below the optimal depth. Only in the longer (250 ft, 76.2 m) span designs the deflection exceeded the optional limit at the optimal depth (see Fig. B.8). For sections that exceed the deflection criteria, increasing the bottom flange positive-moment plate can efficiently reduce the deflection.

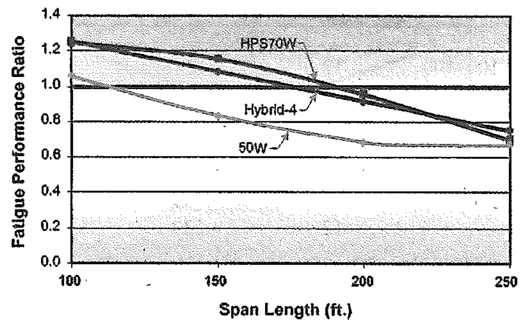


Fig. B.9: Load-induced fatigue (Bottom category C′, 12ft girder spacing)

For load-induced fatigue, the fatigue resistance – the limiting base metal stress range – is a variable that depends on the type of detail under consideration and the structure's average daily truck traffic (ADTT). For typical welded plate girders, detail categories C' and C (corresponding respectively to a welded cross frame stiffener on either flange and to welded shear connectors on the top flange) are likely to be the determining factor and so were considered in the study.

Fig. B.9 plots the fatigue performance ratio (the ratio of the maximum bottom flange fatigue stress range to the 6 ksi (41.4 MPa) allowable value) against span length for homogeneous grade 50W, homogenous HPS 70W, and hybrid-4 material combinations. Values greater than 1.0 indicate that the fatigue limit was exceeded.

The results show that fatigue governed the design of HPS combinations in the 100 ft and 150 ft spans, whereas the 200 ft and 250 ft (61.0 and 76.2 m) spans were not influenced. Note that fatigue also governed the grade 50W designs in the 100 ft. (30.5 m) span lengths but to a lesser extent. The graphs show that the effect of fatigue decreases with increasing span length for all material combinations. The fact that shorter spans are more affected by fatigue largely derives from the lower proportion of dead-load stress to total stress, which in turn results in higher live-load stresses and stress ranges. Typically, the proportion of dead-load stress to total stress increases with span length and girder spacing. Correspondingly, the portion of live-load stress decreases with increasing span length and girder spacing and results in lower stress ranges and less pronounced fatigue effects.

Fig. B.4 and Fig. B.5 show the effects of fatigue on the weight versus web depth curves for the 100 ft and 150 ft (30.5 and 45.7 m) spans. Table B.1 also illustrates the weight savings that HPS 70W designs offer over 50W designs at optimal depth when fatigue is considered.

One of the objectives of the study was to determine where the HPS 70W steel would be cost effective relative to conventional grade 50W steel. To determine the relative fabrication cost of the various designs, steel fabricators were asked to submit fabrication cost estimates for various bridge designs.

Since the study was concerned with relative costs of the two steel types, items that would have no bearing on the relative cost were not included in the estimates, in this way minimizing the overall effort required. The fabricators were requested to estimate the basic girder fabrication costs and to include such items as plate material and shipment costs from the mill; shop fabrication costs, including cutting, welding, handling;

and material waste. The cost of nondestructive testing (NDT), shear connectors, cross frames, field splices, protective painting at girder ends, and transport to the job site, as well as profit, were excluded. The cost of items such as cross frames and shear connectors was expected to contribute to the total cost in the same proportion for both HPS and conventional steel.

The unit costs derived from the fabricator estimates were applied to the weight versus depth graphs for all spans and girder spacing included in the study. In addition to the 50W and HPS 70W curves, the five hybrid alternatives were compared to establish an optimal hybrid configuration with respect to cost.

Fig. B.10 shows a typical set of cost versus web depth curves comparing the two homogenous and five hybrid combinations. These were plotted for 200 ft (61.0 m) span configuration with 9 ft (2.7 m) girder spacing as an example. Similar results were found for other bridge configurations. It became apparent that all hybrids were more cost effective than homogeneous HPS 70W and that the hybrid-4 alternative was the most cost effective in the range of spans and cross sections considered. This hybrid-4 alternative has grade 50W for all webs and positive moment top flanges and HPS 70W for negative-moment top flanges and all bottom flanges.

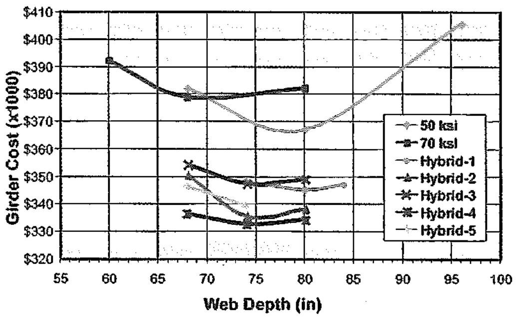


Fig. B.10: Girder cost versus web depth (Typical comparison of hybrid alternatives)

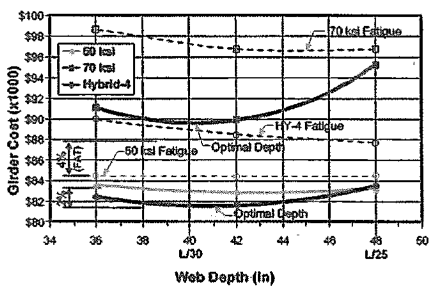


Fig. B.11: Girder cost versus web depth(100 ft. spans and 12 ft. girder spacing)

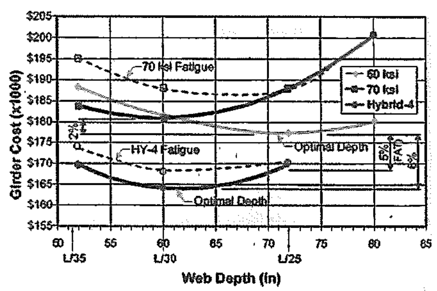


Fig. B.12: Girder cost versus web depth (150 ft. spans and 12 ft. girder spacing)

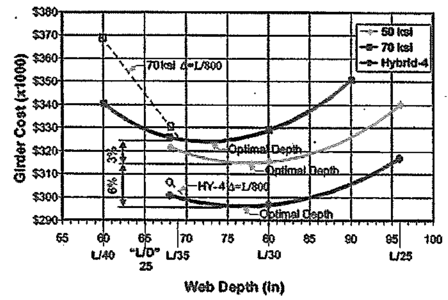


Fig. B.13: Girder cost versus web depth (200 ft. spans and 12 ft. girder spacing)

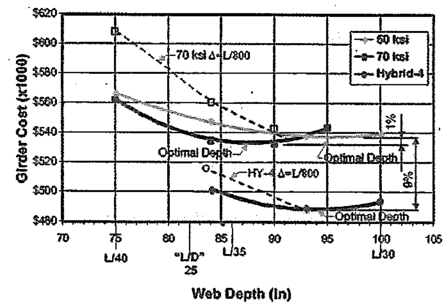


Fig. B.14: Girder cost versus web depth (250 ft. spans and 12 ft. girder spacing)

Fig. B.11 through Fig. B.14 show the relative cost versus web depth graphs for all four span configurations with 12 ft (3.7 m) girder spacing. To simplify the amount of information shown, only the cost-effective hybrid-4 option was included with the two homogeneous options. These graphs also show the relative cost effects of fatigue limitation for 100 ft and 150 ft (30.5 and 45.7 m) spans and deflection limitations for the 200 ft and 250 ft (61.0 and 76.2 m) spans, as previously discussed. Similar results were found for the 9 ft (2.7 m) girder spacing. Table B.2 further summarizes the cost savings of the various configurations at optimal depth based on the cost versus depth curves. Negative percentages in the table indicate costs greater than those for 50W.

At optimal depths, the study shows that homogeneous HPS 70W designs are typically approximately 3% more expensive than the 50W designs. This variation is greater, 8% for the shorter (100 ft, 30.5 m) spans. For the longer (250 ft, 76.2 m) spans with 12 ft (3.7 m) girder spacing, homogenous HPS 70W is actually more economical than 50W by about 1%. Overall hybrid-4 designs are the most economical, conferring savings of 6 to 9% at optimal depth. At lower equivalent web depths, the savings reach 13%.

Cost comparisons for 100 ft (30.5 m) spans with 12 ft (3.7 m) girder spacing show that the homogeneous HPS 70W and hybrid-4 designs confer no cost advantage over grade 50W when fatigue limits control. With fatigue considered at 150 ft (45.7 m) span range, the hybrid-4 designs still maintain a cost advantage over 50W designs by respectively 2 and 5% for the 9 ft and 12 ft (2.7 and 3.7 m) girder spacing. Table B.2 shows adjusted cost saving percentages to accommodate fatigue criteria.

The following conclusions can be drawn from the HPS cost comparison study:

- The most cost effective use of HPS 70W steel was in hybrid designs consisting of grade 50W for all the webs and positive-moment top flanges and grade HPS 70W for negative-moment top flanges and all bottom flanges (hybrid-4).
- Girder designs consisting of all HPS 70W steel were 13 to 20% lighter than conventional 50W steel designs at optimal design depth.

Spans	Girder Spacing	HPS 70W** (%)	Hybrid-4** (%)
100 ft. – 100 ft. (30.5 m – 30.5 m)	12 ft. (3.7 m)	-8 (-14)	2 (-4)
150 ft. – 150 ft. (45.7 m – 45.7 m)	9 ft. (2.7 m) 12 ft. (3.7 m)	-4 (-8) -2 (-6)	4 (2) 8 (5)
200 ft. – 200 ft. (61.0 m – 61.0 m)	9 ft. (2.7 m) 12 ft. (3.7 m)	-4 -3	7 6
250 ft. – 250 ft. (76.2 m – 76.2 m)	9 ft. (2.7 m) 12 ft. (3.7 m)	-3 1	6 9

* Values in parentheses represent cost savings with fatigue taken into consideration.
** Negative percentages indicate costs greater than those for 50W.

*Table B.2: Fabrication Cost Savings at Optimal Depth compared to 50W**

- For bridges that require a structural depth below the optional design depth, HPS 70W and hybrid-4 designs offer even greater savings in weight and cost over homogeneous 50W designs.
- Shallower girder sections can be achieved with HPS 70W steel. The optimal girder depth with this steel is typically less than with 50W steel in all span ranges, although the reduction for 100 ft spans (30.5 m) was minimal.
- Fatigue requirements did influence the performance of the 100 ft and 150 ft (30.5 and 45.7 m) spans. The 200 ft and 250 ft (61.0 and 76.2 m) spans, however, were not affected because of their higher dead-load to live-load ratio. Depending on the volume of the truck traffic, fatigue may dictate the design in the shorter span lengths.
- Deflection criteria (L/800) did control the design and cost of the homogeneous HPS 70W designs at optimal depth for the 250 ft (76.2 m) span length. The criteria did not control the HPS 70W designs at optimal depth for the 200 ft (61.0 m) span length, but depths below the optimal were influenced.
- At optimal depth, the hybrid-4 option was not controlled by deflection at any span length. For web depth less than optimal, deflection influenced hybrid-4 designs in the 250 ft (76.2 m) spans. However, even when deflection criteria controlled, the designs were cost effective in relation to the 50W designs.
- While the homogeneous designs consisting of HPS 70W steel were lighter than 50W steel designs, the former were approximately 3% more expensive than the latter at optimal depth.
- In all configurations except 100 ft (30.5 m) span arrangement, hybrid designs proved more economical than 50W designs by 4 to 9% at optimal depth. Greater savings are possible for depths less than optimal.

The savings offered by high-performance steel may be increased by factors not considered in this study, for example, reduced transportation costs, reduced structure depth, lower fill heights, and lower foundation and bearing costs.

APPENDIX C – HIGH-STRENGTH HIGH-PERFORMANCE STEEL FOR LONG SPAN BRIDGES

The benefits and the limitations of the use of HPS steel for conventional bridge types are well documented. However, the use of higher strength HPS for cable supported long span steel bridges is still emerging. There are several key differences in the behavior of cable-supported long span structures that makes the design optimization of these quite different from those of girder and truss type bridges. These same differences make HPS an ideal material of application on these longer span structures. While these advantages make them even more suitable for long span applications, they are yet to be fully exploited for these types of signature large-scale structures.

For example, the traditional live load deflection control of conventional bridge types requires stiffer girders, or heavier sections. The uses of higher strength steels produce more flexible cross sections that produce higher live load deflections. While live load deflection as a design criterion is under review and may change in the future, this limits the realization of maximum potential benefits of the higher strength steels for conventional bridges. Higher strength HPS girders have higher strength to stiffness ratio (F/E) than the traditional grade 50 steels. While this can be a disadvantage in conventional bridge design due to deflection control, it is very desirable for cable-supported bridges due to the following two factors.

First, the stiffness of long span cable-supported structural systems is less dependent on the stiffness of the superstructure, especially for gravity type loading conditions. The controlling live load deflections of long span structures are produced by global loading conditions involving longer length lane loads. While these deflections are highly dependent on the tower stiffness and the stiffness of the supporting cable system, they can be shown to be practically independent of the superstructure stiffness in the range of girder depths used in typical applications.

Second, the internal moments produced in long span bridge superstructures are due to deformations produced by the global loading. Thus the stiffer girder produces higher demand leading to needing a larger girder that in turn increases the demand. The use of HPS provides a means of increasing capacity without increasing stiffness and can provide tremendous advantageous for long span bridges. This is a factor not recognized widely at the present time and it is hoped that a discussion on this topic would bring it the attention it deserve.

Furthermore, as evident by the Charles River Bridge in Boston, MA, HPS can be the material of choice for special applications on long span bridges. This first time application took advantage of HPS for its steel composite tower and all of the cable anchorages. The design provided optimal solutions considering the complex design issues, improving constructability and the visual aspects of some key components. This proved to be, while a-typical, an interesting application of this material in its early stages of its availability in the US. Many of the fabrication aspects were first-time applications that required thorough investigation to establish the feasibility early in the design process.

3 Fatigue Research on High-Performance Steels in Canada

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3.1 Introduction

Compared to conventional structural grade steels, high-performance steel (HPS) provides higher strength, improved weldability, greatly enhanced fracture toughness, and comparable ductility, as well as having "weathering" properties. The improved characteristics of HPS are achieved through lower levels of carbon and other elements, in conjunction with advanced steel-making practices using either quenching and tempering or thermo-mechanical controlled processing. All of these properties make HPS highly desirable for bridge applications. Indeed, although its development spans only the past decade, it is rapidly gaining popularity for use in highway bridges and is becoming more widely available. Although many highway bridges have already been put into service in the United States, Canada has yet to implement this technology in bridges. Nevertheless, it is anticipated that the use of HPS will become common in the Canadian market in the near future.

Owing primarily to its enhanced strength, HPS has the potential to produce considerable cost savings and, in fact, this has been demonstrated in completed projects in the United States. Other benefits include the potential for reduced girder depths. However, several impediments to the effective use of HPS in conventional bridge girders exist, including the potential for global and local instability, excessive deflections, and fatigue failure due to the lighter structures that are possible with higher strength steels. To overcome these limitations, innovative bridge girder designs have been proposed [3.12], [3.13].

It was demonstrated early in the development of HPS technology [3.14] that utilizing the full benefits of HPS with higher yield strengths may not be possible because the fatigue limit state is likely to control the design. To address this limitation, the research focus has been largely on eliminating fatigue details associated with transverse stiffeners in conventional plate girders, particularly in regions subject to large stress ranges. This approach is based on the assumption that the fatigue performance of details made of HPS is the same as for conventional structural steels. The improved weldability of HPS (resulting in the reduced requirement for pre- and post heat treatment and interpass temperature control, a reduction in the likelihood of hydrogen induced cracking, as well as the recent development of new welding consumables suitable for HPS fabrication) that tends to lead to superior weld quality, in addition to the improved properties of the base material itself (including greatly enhanced toughness) created a need to investigate the fatigue performance of HPS systematically. Until recently, research on HPS was mainly investigating the ultimate limit state of HPS members, with close attention paid to their ductility. Relatively little research has been conducted to characterize its fatigue properties.

Research in Canada on HPS has focused primarily on fatigue aspects. In 2000, a research program was initiated at the University of Alberta that has as one of its objectives the characterization of the fatigue performance of HPS through a comprehensive experimental program on HPS plate material from three different heats. In addition, this ongoing project is investigating methods of fatigue life prediction and, in particular, their applicability to conventional details fabricated using HPS. In 2004, a complementary research project was initiated at Queen's University that is aimed at studying experimentally the fatigue performance of welded details on HPS plates. This project will also involve monitoring of the first HPS bridge constructed in Canada. These two research programs are described in the following sections.

3.2 HPS Research at the University of Alberta

3.2.1 Introduction

Based primarily on previous fatigue research on other steel grades, it has generally been assumed that the fatigue behaviour of HPS will be a function only of the fabrication details and the stress range and will therefore be equivalent to that of other structural grades. A limited number of large-scale tests on HPS girders [3.15] have supported this assertion, but only for particular welded details. Significant differences in the chemical composition and mechanical properties of HPS have necessitated a more rigorous investigation at the material level. In order to obtain a better fundamental understanding of the fatigue properties of HPS, a comprehensive and systematic program of material tests were conducted on ASTM A709 HPS 485W steel and on lower yield strength structural steels with a more typical toughness level to characterize their monotonic and cyclic material properties. The research program also includes the investigation of several methods of fatigue life prediction and their applicability to conventional details fabricated using HPS, validated through comparison with fatigue tests on both conventional grade steels and HPS. Since fatigue testing is time consuming and expensive, these analytical techniques provide a useful tool for predicting both the crack initiation and propagation stages of fatigue life from cyclic material tests. Additional details about the experimental program and the associated fatigue life prediction investigation are presented by [3.4], [3.5].

3.2.2 Experimental Program Overview

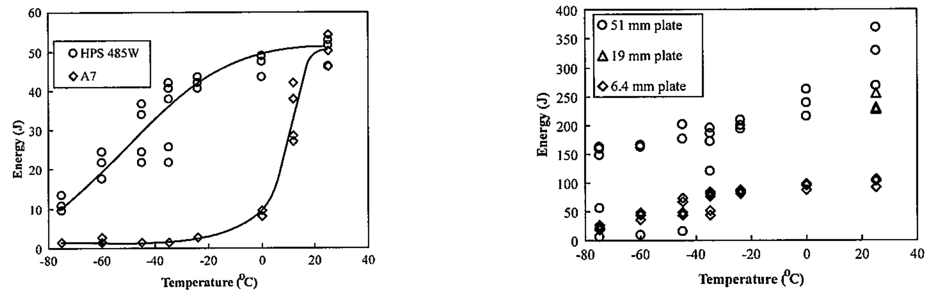
The experimental program was designed to investigate a number of fundamental material properties of HPS that are related to fatigue performance, both in the crack initiation and crack propagation stages, and to provide a comparison of HPS with other structural steel grades. To achieve the latter, similar tests were also conducted on

a 6.4 mm thick ASTM A7 steel plate, a grade commonly used in older structures in North America, to provide a stark difference in properties and, thereby, a basis for comparison. Three HPS 485W steel plates of different thickness (6.4 mm, 19 mm, and 51 mm)-and representing different heats – were used as the HPS source materials. All three heats were early HPS heats produced with the thermo-mechanical controlled process. The 6.4 mm and 51 mm HPS plates were found to have the lowest and the highest toughness levels, respectively, among the three thicknesses. In addition to chemical analyses, Charpy V notch tests, and standard tension coupon tests, a total of 41 fatigue tests were conducted on polished coupons to achieve a number of objectives. Cyclic stress vs. strain properties were obtained by conducting fully reversed, constant strain cyclic tests (FR series) in a closed-loop servo-controlled universal testing machine. The approximate level of the fatigue limit was also determined under a fully reversed, stress-controlled condition (FL series). Some fatigue material specimens were oriented transverse and others parallel to the rolling direction.

3.2.3 Chemical Composition, Fracture Toughness, and Tensile Properties

Chemical analyses revealed that the compositions of the HPS 485W and A7 steels investigated fell within the limits specified in the associated material standards. The HPS had lower carbon and sulphur contents than the A7 steel, and higher contents of alloy elements such as copper, nickel, chromium, and aluminum.

Test results for the Charpy V notch specimens are presented in Figure 3.2.1. As expected, the Charpy V notch energy vs. temperature curves for the HPS 485W and A7 steel specimens (both tested as half of standard size due to the small plate thicknesses) are markedly different, as shown in Fig. 3.2.1(a). Although there was no significant difference revealed in upper and lower shelf energy between the two steels, the ductile-to-brittle transition temperature, taken at half of the upper shelf energy, is significantly lower for HPS 485W than for A7 steel (–50°C for HPS 485W compared to 12°C for A7). Charpy impact tests conducted on specimens obtained from the three plate thicknesses, also corresponding to three different heats of steel, are presented in Fig. 3.2.1(b). In this figure, the energies for the 6.4 mm plate specimens have been adjusted for consistency with the full-size specimens used for the other two plates. There is a large difference in toughness among the different heats of HPS and the 6.4 mm plate seems to have the lowest toughness, while the 51 mm plate has the highest value. Charpy tests on the 19 mm plate, which has the intermediate toughness, were conducted only at room temperature. The fatigue test results presented below were obtained from specimens taken from the 6.4 mm and 51 mm plates, identified as HPS LT and HPS HT, respectively. The suffix represents the relatively low and high toughness character of the two HPS plates.



(a) HPS 485W and A7 steels (6.4 mm plate) (b) HPS 485W steel

Fig. 3.2.1: Charpy V-notch energy vs. temperature

The static tensile tests of the HPS 485W steels revealed that the 0.2% offset static yield stresses were slightly below the nominal value and the mean ultimate stresses were 647 MPa and 518 MPa for the HPS-LT and HPS-HT steels, respectively. Although the ductility of HPS 485W was not as high as that of A7 steel, it was considered to be very good, with at least 23% elongation over a 50 mm gauge length at rupture and 44% reduction of area. Both HPS plates satisfy the material standard requirement, which is 19% elongation.

3.2.4 FR Series Fatigue Tests

Standard flat sheet test specimens were machined from the 6.4 mm and 51 mm HPS plates. The specimens were machined with a reduced section of 8.3 mm wide by 6.0 mm thick and a gauge length of 10 mm was used. The specimens were polished and stress-relieved at 593°C (1100°F) for 2 hours to relieve the machining residual stresses. Specimens were fatigue tested under strain amplitudes varying from 0.1% to 0.625%. Failure was deemed to have occurred when the tensile load had dropped to 50% of its measured initial value. Tests that did not lead to failure of the test specimen were stopped at 10^7 cycles, which is defined herein as a run out. During the initial stage of fatigue testing the material response varied with the number of cycles, but stabilized after approximately 500 cycles. A stable stress vs. strain behavior was reached after 1% to 25% of the total fatigue life.

The cyclic stress vs. strain curves are obtained by joining the tips of the stable hysteresis loops obtained at various strain amplitude levels. A comparison of the monotonic and cyclic stress vs. strain properties of HPS 485W and A7 steels, presented in Figure 3.2.2, indicates different strain response. At small strain ranges A7 steel cyclically softens, whereas HPS 485W cyclically hardens slightly which increases the maximum tolerable stress it can sustain during high cycle fatigue. The test results indicate that HPS 485W has some benefit for high cycle fatigue conditions in terms of strength.

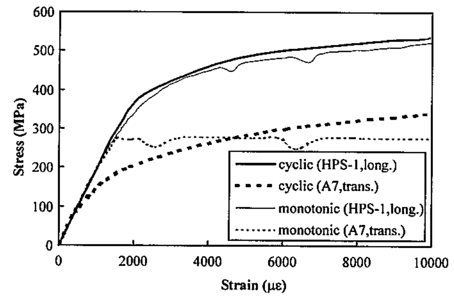
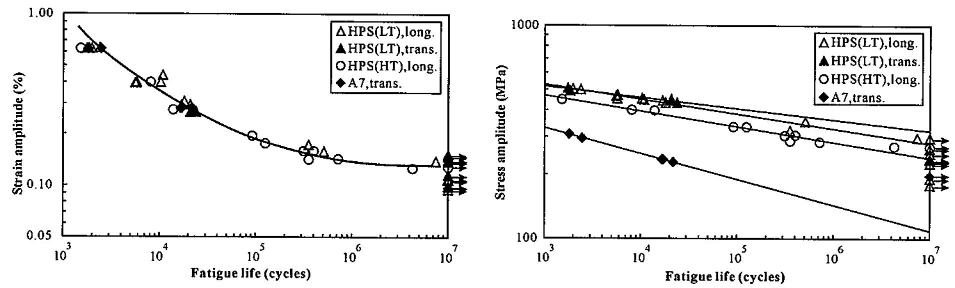


Fig. 3.2.2: Cyclic and monotonic stress vs. strain curves

The fatigue test data expressed in terms of total strain amplitude, $\Delta\varepsilon/2$ vs. number of cycles to failure presented in Fig. 3.2.3(a) indicate that the material coupon orientation has no significant effect on the strain vs. life curve. There was also no apparent difference between the fatigue resistances of the two HPS steels and A7 steel. Conversely, plots of stabilized stress amplitude, $\Delta\sigma/2$ obtained at half of the fatigue life of the smooth specimens, vs. fatigue life, presented in Fig. 3.2.3(b), reveal that although the difference between the transverse and longitudinal fatigue properties for HPS LT is small, there is a significant difference in fatigue life between the HPS and the A7 steel. Although this seems to contradict the observation made from the strain-life relationship, the significant difference observed in the stress-life relationship is caused principally by the large difference in strength between the two steels.



(a) Strain amplitude vs. fatigue life (b) Stress amplitude vs. fatigue life

Fig. 3.2.3: Fatigue Life Data

3.2.5 FL Series Fatigue Tests

The fatigue endurance limit, defined as the stress amplitude (or range) below which fatigue life is infinite, was estimated under a fully reversed stress-controlled condition, whereby the stress amplitude was adjusted based on each successive test result in an attempt to converge to the correct value. Eleven tests were conducted for HPS 485W steel, six on the 6.4 mm plate and five on the 51 mm plate. The test specimens were identical to the FR series specimens.

The test results obtained from the fatigue limit (FL) series are presented in Fig. 3.2.4 for HPS 485W steel. The figure also includes some data from the fully reversed, strain-controlled tests (FR series). These combined results, which incorporate stress ratios, $R = \sigma_{min}/\sigma_{max}$, varying from –0.69 to 0.52, can be used to estimate the fatigue limit. For the HPS LT steel, the fatigue limit is found to lie between 265 MPa (largest stress amplitude with no failures) and 321 MPa (smallest stress amplitude with no run outs). Two run outs and one failure at 7.4×10^6 cycles were observed at a stress amplitude of 298 MPa, which indicates that the fatigue limit is likely close to 300 MPa. On the other hand, the fatigue limit for the HPS HT steel lies below 285 MPa, with two failures at less than 1 million cycles at stress amplitudes of about 285 MPa. The fatigue limit can reasonably be estimated at 270 MPa, with one run out and one specimen that failed at 4.3 million cycles near the lower grip. The difference between the fatigue limits of the two HPS steels can likely be attributed to the difference in tensile strength between them. While the yield strengths for the two steels are approximately the same, HPS HT has a much lower tensile strength than HPS LT (518 MPa and 647 MPa, respectively). As the tests were conducted on polished specimens, a surface roughness correction factor can be applied to obtain the endurance limit of AASHTO [3.1] fatigue Category A details (hot rolled smooth details within 0.025 mm surface smoothness). The stress amplitude endurance limits for both HPS plates are approximately 200 MPa, whereas the corresponding value in the AASHTO design specification is 82.5 MPa (165 MPa if expressed as a stress range). This indicates that HPS may provide a distinct advantage over conventional structural steels in the high cycle fatigue range.

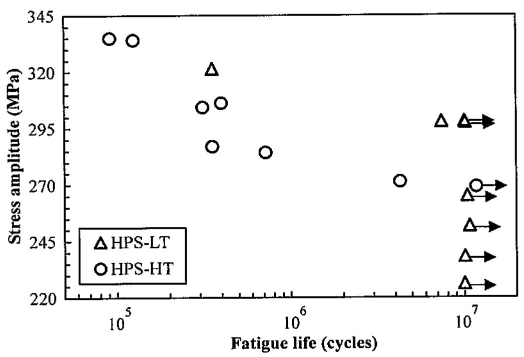


Fig. 3.2.4: Smooth specimen fatigue limit test results

3.2.6 Fatigue Crack Growth Rate Tests

Crack growth rate tests were conducted on the 6.4 mm HPS 485W steel to obtain the steady state crack propagation properties. Single-edged tension specimens were used. The specimens were 50 mm wide, 300 mm long, and 6 mm thick. A 10 mm straight through-thickness notch was made by electrical-discharging machining (EDM). Three specimens each were tested under load ratios, R, of –1 (fully reversed), 0, and 0.5. The load ratio was kept constant for both pre cracking and testing. An electronic imaging system was employed to monitor the crack length visually.

The crack growth rate test results in terms of the crack tip stress intensity factor range, ΔK, for HPS LT steel are presented in Fig. 3.2.5, together with their mean regression lines. The compressive portion of the load cycle is included in calculating

ΔK for R = –1. The mean regression lines are essentially parallel to each other, but for a given ΔK, an increase in R results in an increase in the crack growth rate. When R is increased from 0 to 0.5, the growth rate increases by approximately 80%. The results of tests on 350WT steel (CSA 2000 [3.6]), commonly used for bridge applications in Canada due to enhanced toughness properties, for R = 0.1 are also shown in Fig. 3.2.5. A comparison between the two steel grades indicates that the crack growth properties of this particular HPS are slightly better than for 350WT steel. When compared with values from the literature (Barsom and Rolfe 1999 [3.3]), both the results from 350WT and those from HPS LT at R = 0 are within the general scatter band for ferrite-pearlite steels.

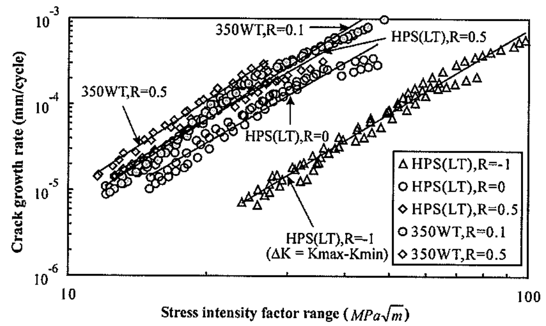


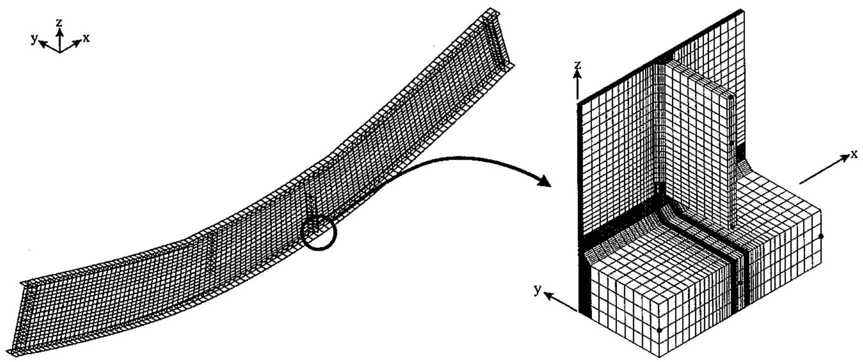
Fig. 3.2.5: Fatigue crack growth rate test results

3.2.7 Fatigue Life Prediction

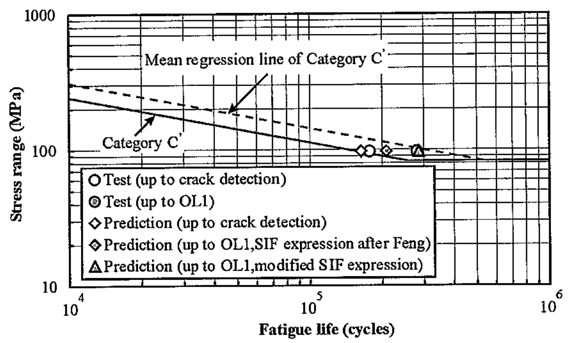
The main objective of the material tests described above is to obtain material data that can be used to predict the fatigue life of typical details so that the fatigue performance of several HPS details can be investigated at minimal experimental effort. A procedure combining the strain-based approach for fatigue crack initiation prediction, fracture mechanics in the crack growth portion of the fatigue life, and finite element analysis to obtain the stress and strain parameters for fatigue life calculations was developed for HPS and validated using test results on fatigue details of both conventional steels and HPS. Details of the fatigue analysis procedures implemented for this phase of the work have been presented elsewhere [3.4], [3.5].

A typical example of an HPS fatigue detail, consisting of a transverse stiffener welded to an HPS steel plate girder [3.15], is illustrated in Fig. 3.2.6. The finite element model, presented in Fig. 3.2.6(a), is required to determine the localized strains at the critical fatigue detail (i.e., the bottom of the stiffener-to-web weld) and to develop the stress correction factors needed to implement the fracture mechanics approach in the crack propagation stage. The predicted fatigue life and test results are both presented in Fig. 3.2.6(b), along with the mean regression and AASHTO ([3.1]) fatigue design curves for this detail. Because the test specimens were periodically overloaded after cracks were detected, the predictions and test results presented in Fig. 3.2.6(b) are only up to the first overload (OL1). It should also be noted that the stress intensity factor (SIF) needed for fatigue crack propagation calculations was obtained both from expressions proposed by [3.8] for a similar detail and from a finite element analysis using the model presented in Fig. 3.2.6(a) for the specific detail geometry tested by

[3.15]. Predictions using both approaches are presented in Fig. 3.2.6(b). Excellent correlation between the test results and the predicted fatigue life is observed for this complex detail.



(a) Finite element model of stiffened plate girder and fatigue detail



(b) Fatigue life prediction and test results

Fig. 3.2.6: Transverse stiffener detail

3.2.8 Observations from University of Alberta Research

The results from this comprehensive experimental investigation of the fatigue properties of HPS at the material level and the development of the fatigue life prediction procedure have led to several observations that can be summarized as follows:

1 The ductility of HPS 485W steel tested under monotonic tension is comparable to the ductility of conventional structural grade steels;
2 An early heat of HPS 485W steel shows similar upper shelf energy absorption as A7 steel, but has a significantly lower transition temperature;
3 Considerable differences in toughness were observed between different heats of HPS;
4 The crack initiation properties of a higher toughness HPS (HPS HT) do not seem to be better than those of the lower toughness HPS (HPS LT), nor are the fatigue properties of HPS significantly superior to those of low toughness conventional steels at higher strain amplitudes;

5 A comparison of crack propagation properties of HPS and 350WT steel indicates that the HPS behaves similarly to conventional grades of structural steel, although it may be marginally superior.
6 The HPS 485W steel tested provides a significantly higher fatigue limit for the base material alone than conventional structural steels, providing a potential advantage in high cycle fatigue applications, although full-scale tests on particular details have indicated that this may not translate into a higher fatigue limit for welded details; and
7 A fatigue life prediction method has been developed and validated that will be used in this ongoing research program to evaluate the fatigue performance of a variety of HPS details.

3.3 HPS Research at the Queen's University

Extensive research conducted in the 1960s suggests that the yield strength of steel does not have a beneficial effect on the fatigue of welded steel details. Fatigue design curves in Canadian, American, and European design standards have been based on this premise. Structural details are classified on the basis of the severity of the stress concentration, ranging from Category A (least severe) to Category E' (most severe). To date, the testing of welded HPS details has been limited and it has been assumed that the current fatigue design categories apply to HPS. [3.9] describe tests of Category B and C HPS details. The results were found to lie within the scatter of their respective weld detail categories. On the other hand, [3.11] report that the fatigue performance for welded details of steels with yield strengths greater than 800 MPa may in fact be worse than that of mild steels, especially for fatigue lives greater than 2×10^6 cycles. [3.2] investigated the fatigue performance of Category C welded details fabricated from a high toughness steel with a nominal yield strength of 800 MPa and subjected to variable amplitude load histories. This work indicated that constant amplitude fatigue data may over-estimate the fatigue life of high strength steel welded details when they are subjected to realistic load histories.
Given the limited data for HPS and other high strength steels in fatigue, it is suggested that more testing is needed to develop additional fatigue data for HPS welded details. The focus of an experimental research program initiated at Queen's University is the fatigue performance of Category C' welded HPS details in the long life range (10^6 to 10^7 cycles) to determine if the current fatigue limit can be applied to the case of HPS, or potentially relaxed. Future work will investigate the long-life performance of other categories of HPS welded details, as well as the performance of HPS in variable amplitude fatigue and corrosive environments. Bridges are subjected to stress cycles that vary with time and often to corrosive environments due to the application of deicing salts. It is known that using constant amplitude fatigue data can be non-conservative for predicting the fatigue life of a component subjected to variable amplitude loading ([3.10]). Furthermore, corrosion causes small surface pits to develop that are potential sites for fatigue crack initiation, which is of particular importance for bridges in Canada because of the presence of de icing salts that accelerate corrosion. Recent work indicates that in a corrosive environment the fatigue limit can be significantly reduced ([3.7]). Understanding the effects of these parameters on the performance of

HPS will provide designers with important guidance in using this material in Canadian bridges. It is anticipated that this research will be complimented by data obtained from the first Canadian HPS bridge to be constructed in Québec.

3.4 First Canadian HPS Demonstration Bridge

The first Canadian highway bridge to employ HPS will be constructed as part of Autoroute Jean Lesage over the Rivière Henri by the Ministry of Transport of Québec. The bridge, which will carry two lanes of traffic, is a skewed four girder structure that has a single span of 47.5 m. The girders, fabricated from HPS 485W steel, are 2010 mm deep unequal flange built up three-plate girders that act compositely with a concrete deck reinforced using fibre-reinforced polymer (FRP) bars. The top flange varies from a 25×450 mm to a 35×450 mm plate and the bottom flange varies from a 35×600 mm to a 50×600 mm plate, each flange having two thickness transition sites. One of these sites also serves as a field girder splice location. The webs of the girders are 14 mm thick plates stiffened using conventional 16×200 mm transverse stiffeners. This bridge was designed using HPS 485W girders primarily to gain experience with this material and to assess its advantages and disadvantages in the Canadian climate and market. Based on the experience gained and an economic evaluation of the completed project, the future of HPS in Canadian bridge construction can be assessed. It is anticipated that the frequency of use of HPS for highway bridges will increase in the near future.

3.5 Summary and Conclusions

High-performance steel research in Canada has focused primarily on fatigue behaviour. Research at the University of Alberta has investigated experimentally both the crack initiation and crack propagation properties of HPS, as well as characterizing other material properties such as static strength, toughness, and chemical composition. It was found that although HPS does not generally exhibit better fatigue performance than conventional structural steels, there is a potential benefit at the fatigue endurance limit, although more research is needed to verify this observation. In addition, a fatigue life prediction model has been developed, and validated using experimental results, that provides a tool for studying analytically the fatigue performance of a variety of HPS details. This analytical investigation is ongoing. Further research has begun at Queen's University to study experimentally the performance of welded HPS details. To complement these two research programs, an HPS demonstration bridge that will be constructed by the Ministry of Transport of Québec will be monitored for fatigue performance by researchers at Queen's University.

3.6 References

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4 High-Performance Steels in Japan

4.1 Bridge High-Performance Steel (BHS) Concept

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Steel materials have been used for bridges for 150 years. Remarkable developments in iron- and steel-making technologies over the years have enhanced the properties of the materials dramatically and increased their applications for structural purposes. In addition, actual applications in structures such as bridges have continuously required higher performance steel materials.

Materials with a great variety of excellent properties have been developed recently. These properties improve the performance of bridges and help to reduce greatly their construction costs. We examined the required performance of bridges to identify what properties are needed in steel materials for bridge applications. As a result, we recommend "BHS" (bridge high-performance steel).

BHS must have various performance properties. The most important of all is high strength, of which BHS has already reached the required level. Today, BHS refers to a steel material with enhanced fracture toughness, weldability, fabricatability, formability and weathering resistance, in addition to excellent yield strength and tensile strength.

Bridge construction costs are divided into material costs, fabrication costs, transportation costs, and erection costs. The stronger the steel material, the lighter the structure, reducing not only the required amount of material, but transportation and erection costs as well. However, with conventional high-strength steel materials, the increased amount of carbon and alloy required for higher strength results in reduced weldability, fabricatability and formability. To make up for such shortcomings, fabrication costs, especially welding costs, have to be raised. Therefore a key issue is how to fabricate high-strength steel at the same cost as mild steel.

Especially high fatigue-resistant performance is required when high-strength steel is used for bridges. In other words, the fatigue strength of the welded structures does not depend on the strength of the steel material. In some cases, the fatigue strength of welded structures using high-strength steel becomes lower than that of mild steel, indicating an inverse relationship. Improving the fatigue strength of welded structures is extremely important in order to use high-strength steel in bridges.

One of the weak points of steel bridges is damage due to corrosion. Paint has been used on conventional steel materials to prevent corrosion. However, the average service life of paint is only 10 years and repainting costs account for a large portion of the life cycle cost (LCC) of steel bridges. Weathering steel was developed in the 1960s and has been widely used to reduce repainting costs. However, the material has not performed effectively under certain environmental conditions. Another urgent issue is the development of steel materials that are much more weathering resistant.

Lamellar tear resistance is especially important among the weldability properties for steel plates for bridges. High stress is often caused in the plate thickness direction in

bridge structures that are three-dimensional in form. A lamellar tear caused by welding may result in unexpected fatigue damage.
Combining steel materials manufactured with the latest technologies and the latest bridge designing and manufacturing technologies is a good way to realize more rational bridge structures and more economical bridges. Defining the properties of steel materials on the basis of performance required for bridges will also contribute greatly to the rationalization of bridge construction.
By specifying that materials with highly improved properties be used in BHS, goals have been clarified involving materials, design, fabrication, maintenance and control. Today the effect of BHS has started to appear in the form of improved efficiency in technological development.
BHS is not just a type of high-performance steel material. Rather, it was created under the concept of a "packaged system" encompassing various technologies such as design, fabrication, and maintenance. It is designed to cut the total costs for steel bridges, including life cycle cost.

4.2 New High-Performance Steel Material for Bridges in Japan

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4.2.1 Application of Conventional High-Performance Steel Materials for Bridges in the Past

Much effort has been done for the use of high-strength steels for bridges in Japan. Tensile stress 600 MPa class steels were first used in bridges in 1960 and were standardized as JIS SM58 (now it is SM570) in 1966. The application of tensile stress 800 MPa class steels was first made in 1964 and since 1969 it has been frequently used for long-span bridges [4.22].
The application of high-performance steels to bridges in Japan started in 1996, when they were partly incorporated into the Specifications for Highway Bridges [4.1]. The production amount of the high-performance steels gradually increased and reached 22% of all bridge steel produced in 1999 as shown in Fig. 4.2.1. The ratios of the production amount of each HPS to all conventional HPS's in 1999 are shown in Table 4.2.1. Weathering steel represents almost 70% of all HPS's produced and is equivalent to 15% of the total bridge steel in 1999. This is followed by steel with specified constant yield strength through heavy gages (15% of the HPS) and steel with low Pcm (11% of the HPS), see Table 4.2.1. Although less than 1% of the total, approximately 2,000 tons a year of the advanced weathering steels [4.1] for high chloride level environments is produced. Nearly 60% of all high strength steels produced in 1999 were yield strength 355 MPa steel (SM490Y), and the production amount of yield strength 450 MPa steel (SM570) steadily increased from 12% in 1996 to 18% in 1999. Steels of yield strength over 450 MPa have not been used since the building of the Akashi-Kaikyo Bridge.
Consequently, weathering resistance, high yield strength and good weldability characterize the latest HPS in Japan, properties which are similar to those of HPS in the USA.

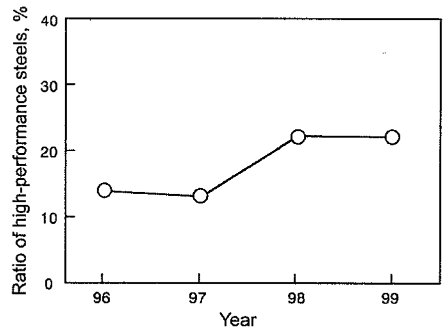

Fig. 4.2.1: Transition of annual production amount of HPS's in Japan

Steel	Ratio (%)
Weathering steel	70
Steel of specified constant yield strength	15
Steel permitting little or no preheat in welding	11
All others	less than 1

Table 4.2.1: Production ratio of each HPS to total HPS's amount in 1999

4.2.2 Proposal of New High-Performance Steel Material for Bridges and Required Properties

The actual application of high-performance steel materials in Japan has grown somewhat as mentioned above, though not markedly. In order to increase applications, Tokyo Institute of Technology SIG1 Project [Special Interest Group 1 in Research Center for Urban Infrastructure, Leader: Prof. Chitoshi Miki] and Japan Iron and Steel Federation have jointly proposed an advanced high-performance steel termed BHS (bridge high-performance steel). There are two types of BHS: one with a yield strength of 500 MPa and the other with a yield strength of 700 MPa, which are respectively called BHS 500 and BHS 700. Additionally, BHS 500W and BHS 700W indicate weathering type BHS. The following are the properties of BHS.

4.2.2.1 Strength Level (Guaranteed yield strength: 500 MPa and 700 MPa)
Table 4.2.2 shows required strength properties. Most bridges are small-to-medium-sized with spans of several tens of meters. A trial design of a bridge with two girders system and a span of about 60m on a road with standard traffic volume was made on the basis of the parameters related to the strength of the steel material. The strength level that can minimize the weight of the steel material is 500 MPa in terms of the yield point (see Fig. 4.2.2). The strength level varies with the stringency of fatigue requirement, dimension of span, and type of bridge. However, this strength may be sufficient for small-to-medium-sized bridges. BHS 500 is designed for bridges in this category.

Steels	Thickness (mm)	Min. YS (N/mm^2)	TS (N/mm^2)	Elongation		
				Thickness (mm)	Coupon Type*	Min. EL (%)
SM570 [4.2] SMA570W [4.3]	t≤16	460	570-720	t≤16	No. 5	19
	16<t≤40	450		16<t	No. 5	26
	40<t≤75	430		20<t	No. 4	20
	75<t≤100	420				
HPS485W [4.4]	6<t≤100	485	585-760	t≤100	GL=50mm	19
BHS500 BHS500W	6<t≤100	500	(570-)	t≤16	No. 5	19
				16<t	No. 5	26
				20<t	No. 4	20
BHS700W	6<t≤100	700	(780-)	t≤16	No. 5	16
				16<t	No. 5	24
				20<t	No. 4	16

*JIS Z 2201 [4.5]
Where, YS is yield strength point, TS is tensile strength and EL is elongation.

Table 4.2.2: Required tensile properties of BHS steels

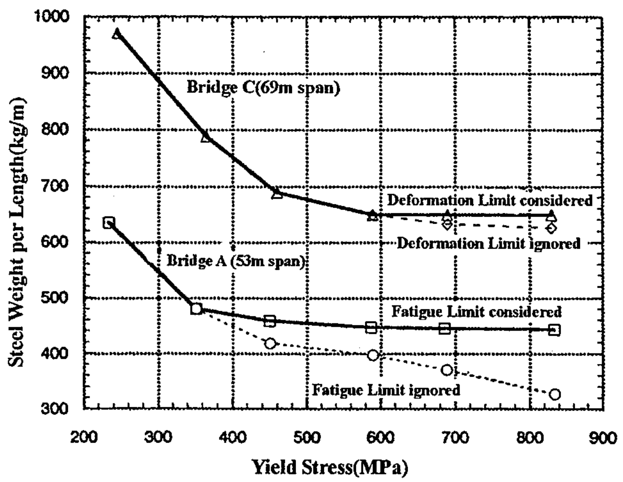

Fig. 4.2.2: Trial design with steel material strength as parameters

On the other hand, when it comes to bridges with long spans employing truss structures that exploit the strength properties of steel material, high-strength steel such as HT780 MPa steel used in the Minato-Ohashi bridge [4.6] is advantageous. BHS 700 is designed for such bridges. At present, the workability (fabricatability, formability and weldability) of BHS 700 is assumed to be equivalent to that of JIS SM 570. With BHS 700 used in sections requiring a plate thickness of 75 mm or more, for example in a bridge with a double girder system and long spans, the thickness can be reduced and the fabrication and inspection processes can be rationalized.
As for the weld metal, the required strength level is the same as for the base metal BHS 500 or BHS 700.

4.2.2.2 Preheating

Preheating has great influence on the fabrication cost of steel bridges; reducing the preheating temperature can lower the cost. BHS 500 has been developed not to require preheating by employing high purity and low Pcm due to the use of thermo mechanical control process (TMCP). The minimum preheating temperature for BHS 700 is set at 50°C by employing the latest TMCP rolling technology.
Table 4.2.3 shows the specified chemical compositions of BHS materials, together with their Pcm (crack parameter, calculated from chemical composition) values, in comparison with conventional JIS SM 570, SMA 570W (weathering steel), and HPS 485W (ASTM A709). Their properties are described below.

Steels	C	Si	Mn	P	S	Cu*	Cr*
SM570 [4.2]	≤0.18	≤0.55	≤1.60	≤0.035	≤0.035	-	-
SMA570W [4.3]	≤0.18	0.15-0.65	≤1.40	≤0.035	≤0.035	0.30-0.50	0.45-0.75
HPS485W [4.4]	≤0.11	0.30-0.50	1.1-1.35	≤0.020	≤0.006	0.25-0.40	0.45-0.70
BHS500	≤0.11	≤0.55	≤2.00	≤0.020	≤0.006	-	-
BHS500W	≤0.11	≤0.50	≤2.00	≤0.020	≤0.006	0.30-0.50	0.45-0.75
BHS700W	≤0.14	≤0.50	≤2.00	≤0.015	≤0.006	≥0.30	0.45-0.80
Steels	Ni*	V	Mo	B	N	P_{CM}	
SM570 [4.2]	-	-	-	-	-	≤0.26/0.27/0.29 2)	
SMA570W [4.3]	0.05-0.30	-	-	-	-		
HPS485W [4.4]	0.25-0.40	0.04-0.08	0.02-0.08	-	≤0.015	-	
BHS500	-	-	-	-	≤0.006	≤0.20	
BHS500W	0.05-0.30	-	-	-	≤0.006	≤0.20	
BHS700W	0.30-2.00	≤0.05	≤0.60	≤0.005	-	≤0.30/0.30/0.32 2)	

*Those amounts may be smaller than the required when the following V value (a new weathering index, mass%)≥1.0.
V=(1.0-0.16[C])*(1.05-0.05[Si])*(1.04-0.016[Mn])*(1.0-0.5[P])*(1.0+1.9[S])*(1.0-0.10[Cu])*(1.0-0.12[Ni])*(1.0-0.3[Nb])*(1.0-1.7[Ti])
PCM(mass%)=C+Si/30+(Mn+Cu+Cr)/20+Ni/60+Mo/15+V/10+5B
Notes:
1) Weathering steels: Enhanced weathering steels of Ni-type are optioned with individual specific chemistries.
2) Corresponding thickness ranges; t≤25 / 25<t≤50 / 50<t≤100
3) BHS700W may need preheating at 50°C

Table 4.2.3: Specified chemical compositions of BHS steels (mass%)

With weldability taken into consideration, carbon is set at 0.11 mass% or lower for BHS 500(W) and 0.14 mass% or lower for BHS 700. Based on this low carbon design, the upper limit of Mn is raised to 2.00 mass% to fully utilize the economic efficiency of the present rolling technology. The upper limit of nitrogen is set at 0.006 mass% by taking into account the strain aging characteristic resulting from today's advanced steelmaking technology. Cu, Cr, and Ni, which are included to make materials weather-resistant, follow the JIS specifications (JIS G 3114) [4.3]. For BHS materials, a new weathering steel alloy indicator V is employed. The value V is set at 1.0 or higher so that both weathering performance and chemical composition design is realized at the same time.
As shown in Table 4.2.3, compared with the conventional SM and SMA steels, the upper limit of Pcm value for BHS 500(W) steel is set at 0.20 to greatly enhance the weldability of the steel material. Though the upper limit of BHS 700W varies with plate thickness, the level is rather low for the strength class, resulting in great improvement in weldability.

4.2.2.3 Cold Forming and Cold Bending (R/t = 7)
Bridge elements are sometimes bent in accordance with the structural design. The radius of such bending work is about 7 times the plate thickness in most cases. The target value for the radius applicable to BHS is set at R/t = 7.
The current Specifications for Highway Bridges requires that Charpy absorbed energy of steel materials be 150 J or higher at a temperature of 0 °C (400 and 500 MPa class steels) or –5 °C (600 MPa class steels). On the basis of a wide range of many years of test data on various types of steel, the value was set at 150 J so that the toughness will remain at least 27 J even if lowered by strain hardening from cold-working. The toughness levels of today's steel materials are very high; materials of 100 J meet the 27-J requirement even after aging.
Table 4.2.4 shows the required toughness of BHS base metal. The Charpy absorbed energy requirement is significantly higher (to 100 J) than for conventional steel materials. The major factors supporting this requirement are the low-carbon composition design as mentioned above, advanced TMCP technology, and the recent achievement of marked reduction of nitrogen, which is thought to have a strong influence on the strain aging characteristic [4.7].

Steels	Thickness (mm)	V-notch Charpy impact test		
		Temp. (°C)	Min. abs. energy (J)	Notch direction
SM570 [4.2] SMA570W [4.3]	12<t≤100	-5	47	Longitudinal
HPS485W [4.4]	12<t≤100	-23	46	Longitudinal
BHS500 BHS500W	12<t≤100	-5	100	Transverse
BHS700W	12<t≤100	-40	100	Longitudinal

Table 4.2.4: Required toughness of BHS steels

4.2.2.4 Measures for High Heat Input Welding (10 kJ/mm)
Demand for high heat input welding in bridges is not as high as in steel-frame buildings. Recently, site electroslag welding in the web for bridges with few girders sometimes requires 15 kJ/mm of heat input. However, high heat input welding is not frequently employed due to the welding workability and the poor quality of results. An examination of various types of welded joints in bridges has shown that a guarantee of 10 kJ/mm heat input is enough [4.8–4.9]. However, the toughness of the heat affected zone and of the weld metal must be carefully considered. Table 4.2.5 shows the permissible welding heat input for BHS 500(W). The lower limit of the absorbed energy in thermally influenced sections after welding is set to 47 J. High heat input welding of up to 10 kJ/mm that satisfies the toughness requirement can be applied. The applicable maximum heat input for BHS 700W is set to 5 kJ/mm by taking into account more stringent toughness levels for improved strength.

Steels	Max. Heat input (kJ/mm)	Required Toughness in HAZ	
		Temp.(°C)	Min. abs. energy (J)
SM570 [4.2]	7	-	-
BHS500 BHS500W	10	-5	47
BHS700W	5	-15	47

Table 4.2.5: Specified available welding heat input for BHS steels

High heat input welding is commonly practiced in Japanese fabrication yards, as it is one of the most efficient welding processes [4.10]. This process was recently used for the site welding of the webs of the triple-girder bridge of the Tokai-Obu Viaduct of Japan High Public Corp. (JHPC). One pass welding by electro-gas welding (EGW) was performed for a 3 m high × 25 mm thick web. The heat input reaches 15 kJ/mm. It is known that when such a high heat input is imposed on ordinary steel, the toughness at the heat-affected-zone (HAZ) deteriorates significantly, mainly due to grain coarsening, that it sometimes becomes lower than the requirement for a base plate. Metallurgical measures [4.11] to prevent HAZ toughness deterioration in a high heat input weld include reducing the carbon equivalent (Ceq) of a steel as shown in Figure 4.2.3, and reducing the contents of impurities like phosphorus and sulfur. In addition, there are special treatments [4.12], such as the TiN, REM-Ti or TiO treatments, which are all effective in preventing grain-coarsening at the HAZ and refining the microstructure for a sufficiently high HAZ toughness in comparison with ordinary steel as shown in Fig. 4.2.4.

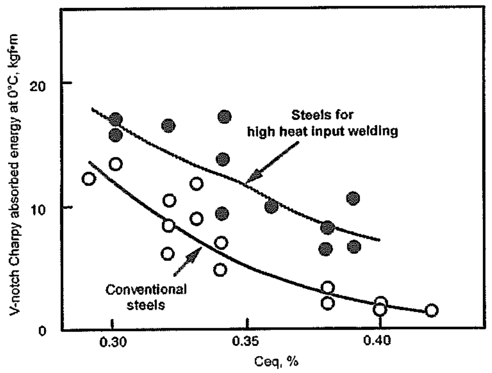

Fig. 4.2.3: Effect of Ceq on HAZ toughness of high heat input welded joint

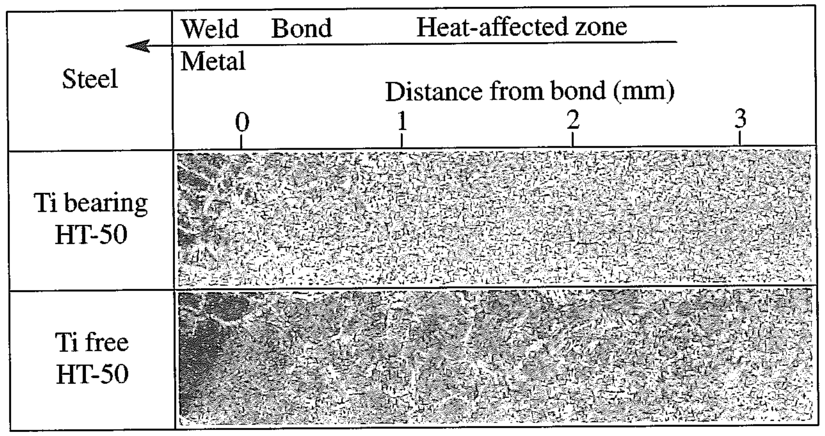

Fig. 4.2.4: HAZ microstructures of high heat input welded joint

4.2.2.5 Performance of Lamellar-Tear Resistance (Z35)

There are many joints in bridge elements to which force is applied in the plate thickness direction. There are also a lot of welded joints that are subject to lamellar tear defects. Lamellar tear defects have been a key issue in connection with the steel bridge piers of elevated roads; various types of measures for the prevention of lamellar tears have been developed so far. Lamellar-tear resistant steel material is specified on the basis of the area reduction rate obtained from a tensile test in the plate thickness direction. BHS material have the highest level Z35 of the specified values for resistance against lamellar tear.

4.2.2.6 Weathering Resistance

Bridges not needing paint have increased in number recently so as to follow the trend for minimum maintenance. In consideration of this trend, the Japanese BHS proposal includes an independent steel material series that is given higher weathering resistance of a level equivalent to that of JIS SMA material. The weathering resistance of BHS 500 is realized by adding alloy elements. BHS 700 material already contains a number

of alloy elements that are designed for improved strength but are effective for weathering resistance as well. Therefore, BHS 700 as it is can serve as a weathering resistant material. For higher levels of weathering resistance, materials will be prepared separately as necessary. We also propose an index to be used in judging the level of weathering resistance necessary with respect to the service environment and types of steel.

4.2.3 Summary

Required properties for steel bridges from the standpoint of design, fabrication and maintenance are discussed in this chapter. Two main types of steels for bridges, BHS 500 and BHS 700, are proposed.

4.3 Manufacturing Technology and Confirmed Performance of BHS

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4.3.1 Progress in Manufacturing Technology that supports BHS

4.3.1.1 Steel Materials of Higher Purity

Steel materials have improved remarkably in terms of performance and high-purity technology in the field of steelmaking. The characteristics of steel materials depend highly on such impurity elements as carbon, sulfur, phosphorous and nitrogen. The demand for fewer impurities is mostly for the purpose of improving the quality of steel materials. However, it is also prompted by the desire to improve processes, including continuous annealing and rolling technique, and prevent surface flaws and establish energy-saving facilities [4.13]. Table 4.3.1 shows the changes in limits related to the reduction of impurities in steel. This table clearly indicates that iron- and steel-making technologies and the purity of steel materials as a result of the development of such technologies have improved greatly since the current JIS SM570 was first specified.

	Year			
Elements	1970	1980	1990	2000
C	56	22	8	3
S	18	6	2	0.5
P	148	41	12	3
O	23	11	5	2
N	30	17	10	5
H	2.0	1.3	0.8	0.5
				(ppm)

Table 4.3.1: Changes in limits related with the reduction of impurity elements in steel

4.3.1.2 Manufacturing Technology for High-Performance Steel based on TMCP

The performance of steel plates was originally improved due to the controlled-rolling method. TMCP (Thermo-Mechanical Control Process) technology, which combines the controlled-rolling method and controlled-cooling method, has contributed to further improvement of the high-strength and high-performance characteristics of steel plates. Fig. 4.3.1 shows the types of rolling technologies practiced today in steel plate production. Today's steel plates are produced not by the ordinary rolling method but by TMCP technology, which incorporates micro-structural control suitable for higher strength and thickness together with advanced micro-alloying technology. Furthermore, the latest rolling technologies include TMCP with either highly controlled cooling or direct quenching and tempering (DQ-T). With these advanced technologies, Ceq (carbon equivalent) and Pcm levels are drastically reduced in comparison with the levels found in the past and the preheating requirement that was a significant burden in the welding of SM570 and HT780 MPa steel is reduced effectively.

The application of TMCP to bridge steel started around 1996 when the HPS began to be used for bridges. The production of HPS's by TMCP is expected to steadily increase with the increase of HPS for bridge applications from now on.

The characteristics of TMCP from a metallurgical point of view are grain refining and formation of a fine bainitic microstructure, which are both induced by the controlled-rolling and controlled cooling process [4.14]. These micro-structural control methods raise the yield strength and the toughness simultaneously so that alloys can be saved by TMCP and the Pcm value can be lowered in comparison with the conventional as-rolled process. As a result, TMCP steels have better weldability than conventional steels.

Because it saves alloys and lowers Pcm value, TMCP offers various advantages to steels such as heavy gaging and acceptance of high heat-input welding as listed in Table 4.3.2. As a remarkable recent example, the TMCP steel of yield strength Gr. 450 MPa (SM570TMC) [4.15],[4.16], which has an extremely low Pcm, has been introduced into the market. The TMCP process is also applied during the rolling process of a yield strength Gr. 685 MPa steel. Some 780 MPa-class steels of low Pcm, which are produced by direct quenching and tempering or the precipitation hardening process, were used in the Akashi-Kaikyo Bridge [4.17].

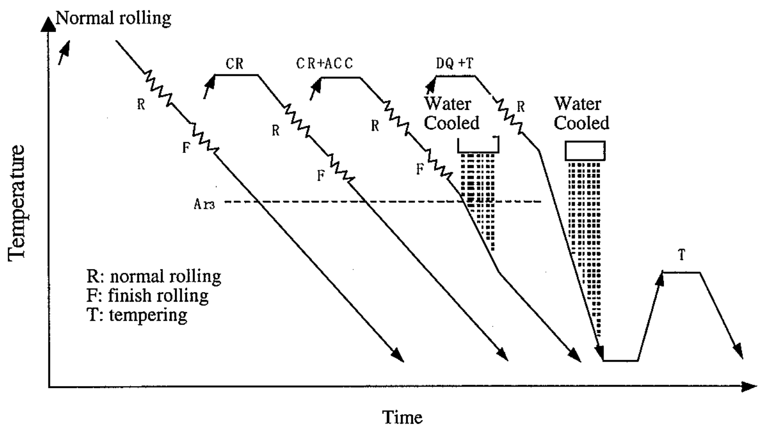


Fig. 4.3.1: Types of rolling technologies in today's steel plate production

Weld Consumable	Hydrogen Content (ml/100g)	Welding Pass	Heat Input (kJ/cm)	Preheat Temp. (°C)	Inter-pass Temp. (°C)	Liquid Penetration Test
GMAW Solid Wire (1.2mm ∅)	(30°C×80%RH)	Root	32	20	20~50	No Cracks
		Filling	17			
	1.9	Capping	10	50	50~150	No Cracks
		Final	8			
GMAW FCAW (1.2mm ∅)	(30°C×80%RH)	Root	32	20	20~50	No Cracks
		Filling	17			
	4.3	Capping	9	50	50~150	No Cracks
		Final	8			
Note : Total Passes; approximately 80 passes						

Table 4.3.2: Window type restraint weld cracking test results for SM570TMC (Pcm=0.17, t=83mm)

4.3.2 Confirmed Performance of BHS 500

Fig. 4.3.2 shows the recorded yield strength of SM570 manufactured with TMCP. The conventional SM570 reached a yield strength of 500 MPa almost satisfactorily. In other words, the BHS steel proposed in Chapter 4.2 was obtained with high-performance engineering technology centering on TMCP as mentioned above to inherit the excellent performance described above. The following are the typical characteristics of BHS 500, which was experimentally produced on the basis of TMCP-manufactured SM570.

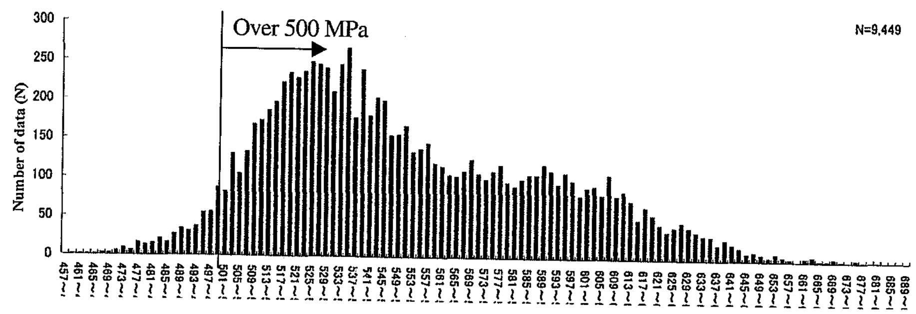


Fig. 4.3.2: Distribution of SM570 steel yield strength, manufactured with TMCP

Table 4.3.3 shows actual measured values of chemical composition with the required target values to satisfy the basic characteristic of experimentally produced BHS. The strength and notch toughness (Charpy value) are improved by using the latest TMCP technology and making optimum use of the microstructure control mechanism that combines micro-alloying, controlled-rolling, and controlled-cooling methods. Basically TMCP can make more effective use of thermal processing in the rolling process than the conventional QT method. The manufacture is carried out through a

combination of advanced process control technologies, including strict setting of conditions as well as the rolling schedule and thorough realization of controlled-cooling conditions on the rolling line. Through the application of such advanced TMCP technology, the carbon content is kept low, achieving a low Pcm level that helps keep the welding process free of preheating.
Besides, a lamellar tear resistance level of Z35 is achieved by reducing the amount of S to the lowest level possible. Furthermore, with fine-grain and high-toughness technology, deterioration of the toughness of HAZ is reduced and high heat input is also permissible.

Steel type	No.	t(mm)	C	Si	Mn	P	S	Cu	Ni	Cr	Pcm
BHS500	Required value		≤0.11	≤0.55	≤1.60	≤0.02	≤0.006	added if needed			≤0.20
	1	22	0.08	0.29	1.56	0.009	0.002				0.17
	2	50	0.08	0.29	1.56	0.009	0.002				0.17
	3	100	0.09	0.29	1.57	0.011	0.002				0.18
BHS500W	Required value		≤0.11	≤0.55	≤1.60	≤0.02	≤0.006	0.30-0.50	0.05-0.30	0.45-0.75	≤0.20
	4	25	0.05	0.21	1.35	0.006	0.002	0.33	0.26	0.47	0.17
	5	60	0.05	0.21	1.35	0.006	0.002	0.33	0.26	0.47	0.17
	6	100	0.04	0.19	1.35	0.003	0.001	0.35	0.26	0.49	0.18

Table 4.3.3: Chemical compositions of some BHS steel plates; required value and examples (mass%)

Fig. 4.3.3 shows the microstructure photos of BHS material and conventional QT type SM570Q. Both basically have bainitic microstructures. In this figure, the BHS manufactured with TMCP has fine granular bainite grains, proving its efficiency in forming and developing fine grains.

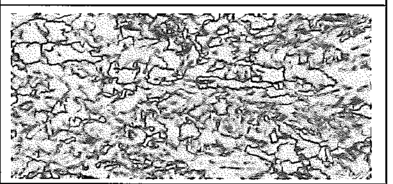
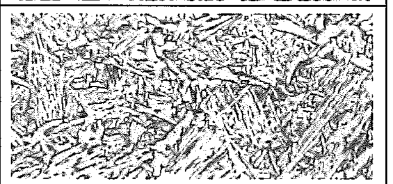
Magnification Position	×100 200μ	×400 50μ
BHS500 t=50 mm (t/4)		
SM570Q t=55 mm (t/4)		

Fig. 4.3.3: Microstructure photos of BHS material and conventional QT type SM570Q

Table 4.3.4 shows the mechanical properties of BHS 500 obtained from various types of test. It indicates clearly that both BHS 500 and BHS 500W satisfy the yield strength and elongation requirements up to a plate thickness of 100 mm. The Charpy values are also large enough. When the material is to be used in a structure that is required to be earthquake-proof, a -50°C temperature shift could suffice for this purpose [4.18]. BHS 500, in particular, with a plate thickness of 100 mm would provide sufficient earthquake resistance.

steel type	No.	thickness (mm)	test position	YS (N/mm²)	TS (N/mm²)	EL (%)		vE-5 (J)
Required value (lower limit)				500		t≤16	19	100
						$16<t$	26	
						$20<t$	20	
BHS500	1	22	¼t	560	645	30		324
	2	50		504	609	32		317
	3	100		534	624	27		291
BHS500W	4	25	¼t	566	646	38		324
	5	60		564	638	26		175
	6	100		517	608	27		174
Where, YS is yield strength, TS is tensile strength, EL is elongation and vE-5 is energy obtained from Charpy test at the temperature of –5 centigrade.								

Table 4.3.4: Mechanical properties of some BHS500(W) steel plates

Table 4.3.5 shows the strain aging properties of BHS 500. The performance at a strain of 10% is higher than the requirement; the material can withstand cold bending of 7 times the plate thickness.

steel type	Strain aging post heat	No.	thickness (mm)	test position	test direction	Before aging	After aging
						vE-5(J)	
Required value (lower limit)					L.C.	100	47
BHS500	10% 250°C×1hr	1	22	¼t	L	324	271
		2	50		L	317	299

Table 4.3.5: Example of BHS500 steel plates strain ageing properties

Since BHS material exhibits a low Pcm value and high toughness, it has the following two advantages. One is an improved cold-cracking characteristic, which may lead to the easing of the requirement of $\sqrt{2t}$ for fillet-weld leg length in the current Specifications for Highway Bridges [4.19]. As a result, it will be possible to weld a leg length subject to a single pass without causing cold cracks, even if the leg length of the longitudinal welding joining flange and web in a two-girder bridge, which usually requires thick plates, exceeds the dimension for a single pass of welding according to the specifications above. The other advantage is high heat input. Applying a high heat input welding method such as electroslag welding to a thick web, such as a two-girder bridge, will help reduce fabrication costs.

Basic cold cracking properties can be identified on the basis of the Pcm values shown in Table 4.3.3. They appear to satisfy the requirement level sufficiently. We conducted a Y-groove weld cracking test (JIS Z3158) [4.20] using BHS 500 steel plate having a thickness of 50 mm and BHS 500W steel plate having a thickness of 25 mm to determine the preheating temperature for preventing cold cracking of the experimental BHS 500 and BHS 500W steels. The preheating temperatures were 16, 50, and 70°C for BHS 500 and 30, 50, and 75°C for BHS 500W. The repetition number at each level was 3. Fig 4.3.4 shows the cross-sectional photos of the test pieces of the Y-groove weld cracking test, while Fig. 4.3.5 shows the results of the test. The minimum preheating temperature obtained from the test is 16°C or less (in an environment with a temperature of 16°C and a humidity of 40%, GMAW: preheating can be omitted). Also in the case of BHS 500W, the minimum preheating temperature obtained from the test is 30°C or less (in an environment with a temperature of 30°C and a humidity of 80%, SMAW: preheating can also be omitted in most cases). There is no related report on plate thicknesses exceeding 50 mm; experiments are necessary to confirm the necessity of preheating with thicker plates.

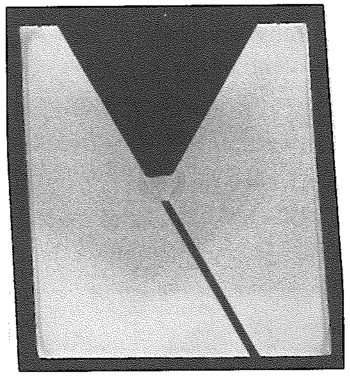


Fig 4.3.4: Cross sectional photo of Y-groove weld cracking test, [4.20]

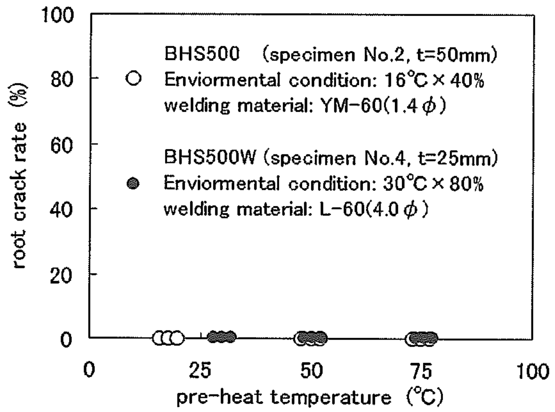


Fig. 4.3.5: Y-groove weld cracking test result

Table 4.3.6 shows the results of the weld joint performance test. Looking at heat input levels of 10 kJ/mm or lower, the performance meets the requirement in all of the joints on BHS 500 (No. 3) having a thickness of 100 mm. For high heat input of 10 kJ/mm or higher, Charpy values are lower, but the performance also meets the requirement level.

Steel type	No.	Thickness (mm)	Welding type	Heat input (kJ/mm)	TS (N/mm²)	VE-5 (J)				
						WM	FL	HAZ1	HAZ3	HAZ5
Required value (lower limit)				≤10kJ/mm		47				
BHS500	1	22	SAW	19.5	582	131	82	74	85	109
	3	100	CO2	3.4	632	120	138	204	220	276
BHS500W	4	25	EGW	19.2	593	97	101	94	220	312
Where, WM is weld metal, FL is fusion line, HAZ1 is position in HAZ 1mm from fusion line, HAZ3 is 3mm and HAZ5 is 5mm.										

Table 4.3.6: Example of welded joint properties of BHS500(W) steel plates

Table 4.3.7 shows the results of the heat-straightening test. The strength properties and toughness are satisfactory even after heat-straightening up to 1000°C. This means heat-straightening can be applied to the material for fabrication.

steel type	No.	Thickness (mm)	Heating temperature	Cooling method	YS (N/mm²)	TS (N/mm²)	EL (%)	VE-5 (J)
BHS500	1	22	900	water	555	641	30	305
			1000		553	644	29	310
	3	100	1000	water	535	641	25	245

Table 4.3.7: Example of local gas-reheat properties of BHS500(W) steel plates

Judging from the results above, BHS 500 and BHS 500W have the same or higher performance as conventional steel materials such as SM490, at least with respect to welding, bending, and heat-straightening. Besides, with these BHS, there is no basis for requiring an increase in the number of processes as for SM570. As mentioned above, the specification of fillet weld leg length $\sqrt{2t}$ will also be reduced.

4.3.3 Design of BHS 700

BHS 700 is still in the experimental stage. It is based on low-preheating HT780 used for the Akashi-Kaikyo Bridge and can be developed into a steel material that guarantees an extremely low C or a yield strength of 700 MPa with TMCP technologies such as DQ-T. In other words, by making use of the microstructure reinforcement mechanism of TMCP technology, the material will be refined and made much tougher, while a low Pcm level for reduced weld preheating needs will be realized as a result of successful reduction of the amount of C. In addition, the toughness deterioration in HAZ will be decreased and high-heat input welding will be possible.

4.3.4 Welding Materials for BHS

The welding of BHS is required to have the same strength as that of the base metal. Toughness is also specified for BHS weld. Table 4.3.8 shows the current status of welding materials for BHS steel. The current welding materials can be used for BHS 500. For BHS 500W, the current welding materials can be applied to general specification and low-temperature specification up to -20°C. However, for BHS 500W -40°C, -60°C, and high heat input specifications, and also for BHS 700W, the current weld materials cannot be applied: they are under study.

Steel type	Specification	Welding type			
		SMAW	GMAW	SAW	EGW
BHS500	Normal	O	O	O	–
	Large heat input[1)]	–	–	O	O
	Low temperature[2)],-20°C	O	O	O	
	Low temperature[2)],-40°C	O	O	O	
	Low temperature[2)],-60°C	O	O	O	
BHS500W	Normal	O	O	O	
	Large heat input[1)]	–	–	Need examination	O
	Low temperature[2)],-20°C	O	O	O	–
	Low temperature[2)],-40°C	Need examination	Need examination	O	–
	Low temperature[2)],-60°C	Need examination	Need examination	O	–
BHS700W	Normal	Need examination	Need examination	Need examination	–

(comments)
1) Plate thickness=15~25mm, for SAW and EGW Heat input ≤ 15kJ/mm
2) Test temperature=-20°C, -40°C, -60°C vE ≥ 47J
3) Meaning of symbols O : possible to use conventional weld material (development is completed),
– : not objective condition

Table 4.3.8: Current status of weld materials for BHS steel

Steel types	Weld Materials	Mechanical properties				Chemical composition (%)						
		σ_y MPa	σ_B MPa	El. %	VE_{-20} J	C	Si	Mn	Cu	Ni	Cr	Mo
BHS500	SMAW	550	650	30	150	0.07	0.61	1.15	–	0.63	–	0.26
	GMAW[1)]	580	660	29	150	0.08	0.50	1.09	0.1	–	0.42	0.29
	FCAW[2)]	560	620	27	100	0.05	0.33	1.13	–	1.00	–	0.21
	SAW	570	660	26	80	0.08	0.34	1.58	0.08	–	–	0.45
BHS500W	SMAW	540	640	29	170 (-5°C)	0.07	0.58	1.02	0.35	0.49	0.57	–
	GMAW[1)]	570	650	26	190 (-5°C)	0.06	0.59	1.09	0.44	0.56	0.56	–
	FCAW[2)]	550	620	27	78 (-5°C)	0.04	0.55	1.14	0.41	0.48	0.52	–
	SAW	540	620	25	120 (-5°C)	0.05	0.32	1.53	0.33	0.18	0.59	–

(comments)
1) Ar+20%CO_2 sealed gas
2) CO_2 sealed gas

Table 4.3.9: Examples of weld materials chemical compositions for BHS 500 and BHS 500W

4.3.5 Summary

Thanks to the new development of steel making process, new high strength steel "BHS" is realized. This steel shows good mechanical properties and weldability, and it can satisfy the required properties given from structural examinations.

4.4 Advanced Weathering Steels

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4.4.1 Performance and Current Application Status of Advanced Weathering Steels

4.4.1.1 Development Objectives

Application of weathering steels to bridges in Japan started substantially in 1964 and has since been increasing steadily as shown in Fig. 4.4.1. Today about 15% of bridges are made of weathering steel.

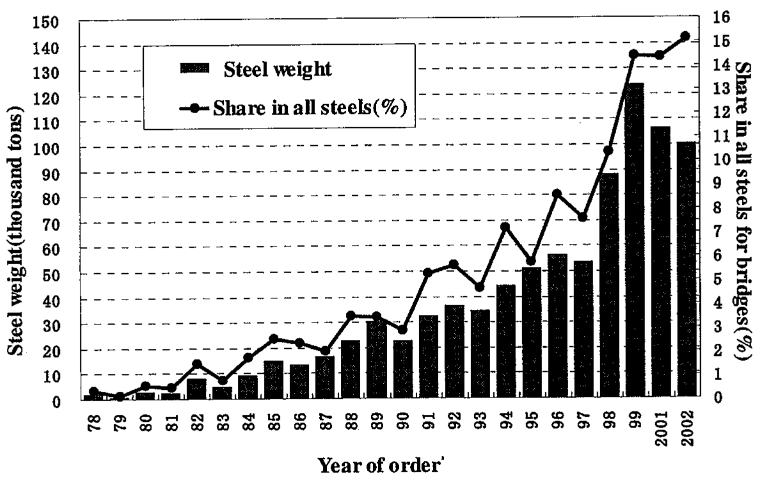


Fig. 4.4.1: Trends in application of weathering steel to bridges in Japan

A joint study on weathering steel [JIS G3114: Hot-rolled weathering steel for welded structures], was conducted from 1980 through 1990 by three parties: Public Works Research Institute of the then Ministry of Construction, Kozai Club and Japan Association of Steel Bridge Construction. In this research, weathering steel specimens were exposed under 41 bridge girders throughout Japan for 9 years (in hostile environments, including the condition that adhered salt content should not be washed away by rain water). The research concluded that weathering steel could be used for bridges where the air-borne salt content was 0.05 mdd (mg/dm^2/day) or less.
The joint research report [4.21] issued in 1993 was titled "Guidelines for Design and Construction of Paintless Weather-Resistant Bridges (Revision Proposal)." This stipulated the air-borne salt content as noted above and also the minimum distance from the shore for exemption from measurement of air-borne salt, dividing Japan into five areas. The information is reflected in the current Specifications for Highway Bridges (see Fig. 4.4.2).

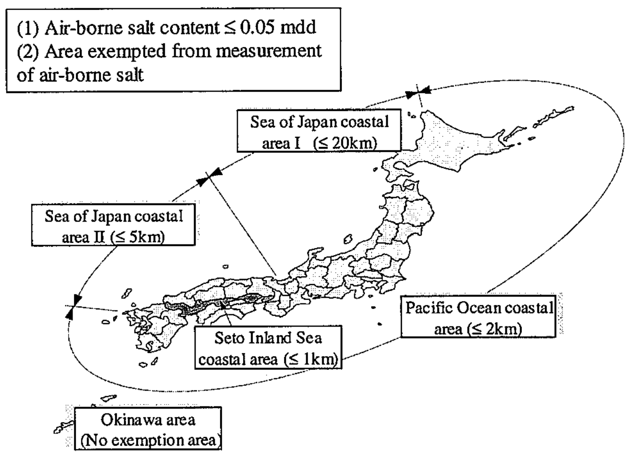


Fig. 4.4.2: Areas where JIS weathering steels could be used according to [4.21]

Fig. 4.4.3 shows that, based on cumulative corrosion to the horizontal exposure test pieces over the 9-year study, very little corrosion is likely to occur, assuming that dry and wet conditions repeat normally over a period of 100 years.
However, the use of antifreeze agents sprayed over the road surface during winter has increased since studded tires were banned in 1991. Fig. 4.4.3 shows the changes. Recently, weathering steel bridges have suffered problems such as excessive rusting caused by splashing or leaking of road surface fluids, including antifreeze agents.

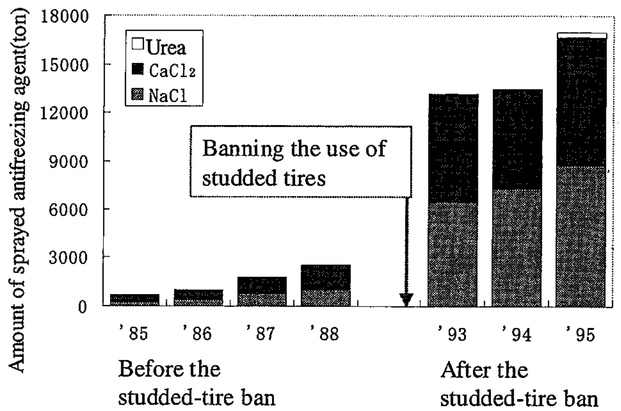
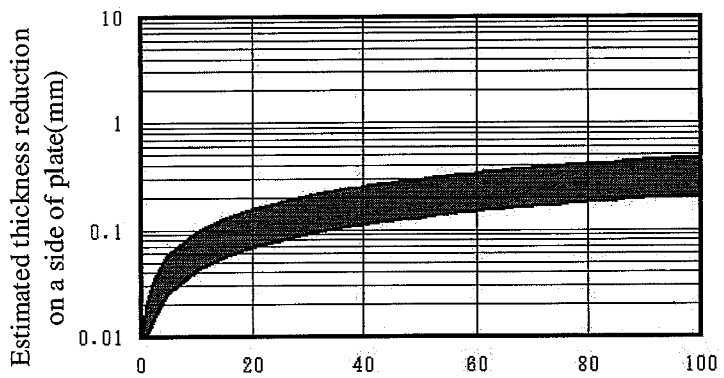


Fig. 4.4.3: Changes in amount of sprayed antifreeze agents for road surfaces (Aomori Prefecture in Japan)

The performance of conventional weathering steels is degraded by air-borne salt in coastal areas and antifreeze agents in winter. Steel manufacturers have therefore developed advanced weathering steels by adding various alloy elements to conventional materials. The following section describes the targets when developing these advanced weathering steels.

1 Applicable environments
The advanced weathering steels developed by various steel manufacturers contain various alloy elements at different contents according to their respective design, aiming to reduce corrosion below the level shown in Fig. 4.4.4 one hundred years after erection in an environment where air-borne salt content exceeds 0.05 mdd. These alloy elements prevent chloride ions from reaching the surface of steel through rust, by helping to form protective rust.



Estimated thickness reduction by amount of air-borne salt (mm)
(Air-borne salt: 0.05 mdd or less)

Fig. 4.4.4: Estimated corrosion of weathering steel

This type of advanced weathering steel, in view of the properties required of steels for welded structures and cost performance as mentioned below, seems inadequate for hostile environments in which seawater droplets directly adhere to the surface of the steel, and so could only be used in certain environments.

2 Mechanical properties of steel materials
The mechanical properties of advanced weathering steels are designed to satisfy the specified values for SMA 400W, 490W, and 570W in JIS G 3114 [Hot-rolled weathering steels for welded structures]; bridges will therefore be designed in the same way as with conventional JIS weathering steels.

3 Weldability of steels
Manual welding, submerged arc welding, and CO_2 arc welding, which are commonly used for bridge-building, can be used for all advanced weathering steels. The chemical compositions of the steels are designed especially for welding without preheating or with reduced preheating.

4 Cost performance of steels
Various advanced weathering steels have been developed to improve the weathering resistance in salty environments, to satisfy the mechanical properties and weldability of steel materials as mentioned above, and also to reduce the cost of the welded structure. Advanced weathering steels cost around 40% more than JIS weathering steels. This increased cost per painting surface area of bridge amounts to an increase of about 3,000 to 4,000 yen/m^2 (on the assumption that the painting area is 15 m^2/ton).

4.4.1.2 Example Performance of Developed Nickel-Containing Advanced Weathering Steels (quoted from "Application of Weathering Steels to Bridges [guidebook]")

1 Chemical composition
Table 4.4.1 shows the chemical composition of advanced weathering steels developed by Japanese manufacturers. The nickel content in the respective types of steel is higher than that in JIS weathering steels, and exceeds the upper limit of the JIS standard of 0.3%. These steels are therefore called "nickel-containing advanced weathering steels" and have the following properties.

Chemical composition type
0.3% Cu-3% Ni type
1.5% Ni-0.3% Mo type
Extremely low C-0.3% Cu-2.5% Ni type
0.3% Cu-2% Ni-0.5% Cr-0.3% Mo type
1% Cu-1% Ni-0.05% Ti type

Table 4.4.1: Chemical composition type of nickel-containing advanced weathering steels

2 Results of atmospheric exposure tests
Fig. 4.4.5 and Fig. 4.4.6 show the results of atmospheric exposure tests for various nickel-containing advanced weathering steels conducted in a salty environment, in comparison with the JIS weathering steels. Because the exposure conditions are different for each type, the results cannot be compared directly. Nevertheless, they indicated higher resistance against salty environments than JIS weathering steels.

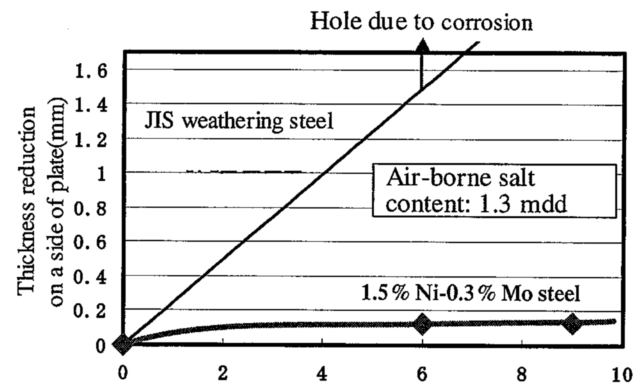


Fig. 4.4.5: Corrosion test results for nickel-containing advanced weathering steel conducted in salty environment (Example 1)

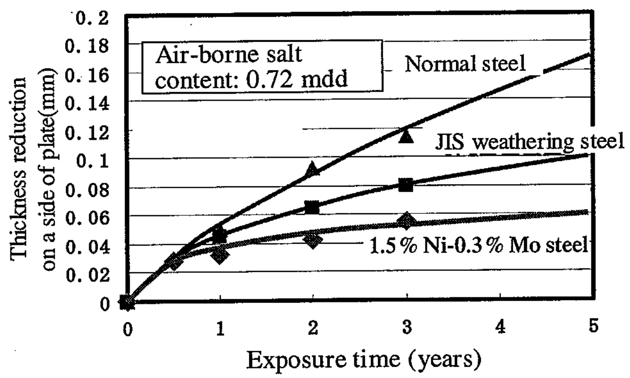


Fig. 4.4.6: Corrosion test results for nickel-containing advanced weathering steel conducted in salty environment (Example 2)

3 Mechanical properties

Table 4-2 shows typical mechanical properties of nickel-containing advanced weathering steels. All of the properties satisfy the specifications of JIS weathering steels and so the materials qualify for use in bridges.

Type of steel	Chemical composition type	Plate thickness (mm)	Tensile test		Impact test	
			Yield stress MPa	Tensile strength MPa	Test temp. (c)	Absorbed energy (J)
Tensile strength 490 MPa class steel	0.3% Cu-3% Ni	25	405	512	0	230
	1.5% Ni-0.3% Mo	50	358	515	0	281
	Extremely low C-0.3% Cu-2.5% Ni	50	428	565	0	357
	0.3% Cu-2% Ni-0.5% Cr-0.3% Mo	25	455	594	0	301
	1% Cu-1% Ni-0.05% Ti	25	474	584	0	322
Tensile strength 570 MPa class steel	0.3% Cu-3% Ni	40	599	666	-5	310
	Extremely low C-0.3% Cu-2.5% Ni	50	492	621	-5	326
	0.3% Cu-2% Ni-0.5% Cr-0.3% Mo	50	549	672	-5	319

Table 4.4.2: Typical mechanical properties of nickel-containing advanced weathering steels

4 Weldability test results

A Y-groove weld cracking test (JIS Z3158) [4.22] was conducted to determine the weldability of nickel-containing advanced weathering steels. As shown in Table 4.4.3, no crack developed in the heat affected zones during welding at room temperature without preheating. The Y-groove weld cracking test involves a high restraining stress and low welding heat input. If the results indicate that preheating is not required, it means the tested material has excellent weldability.

Type of steel	Chemical composition type	Plate thickness (mm)	Results of Y-groove weld cracking test Preheating temperature required for preventing weld crack (c)
Tensile strength 490 MPa class steel	0.3% Cu-3% Ni	25	25
	1.5% Ni-0.3% Mo	50	20
	Extremely low C-0.3% Cu-2.5% Ni	50	20
	0.3% Cu-2% Ni-0.5% Cr-0.3% Mo	25	20
	1% Cu-1% Ni-0.05% Ti	25	25
Tensile strength 570 MPa class steel	0.3% Cu-3% Ni	40	20
	Extremely low C-0.3% Cu-2.5% Ni	50	20
	0.3% Cu-2% Ni-0.5% Cr-0.3% Mo	50	20

Table 4.4.3: Weldability test results for nickel-containing advanced weathering steels

4 Weld joint performance test results

Table 4.4.4 shows Charpy impact test results for weld joints made on nickel-containing advanced weathering steels. Even submerged arc welding and electroslag welding yielded sufficiently high values of absorbed energy in excess of 47 J, a standard requirement.