HIGH TOUGHNESS OF HPS: CAN IT HELP YOU IN FATIGUE DESIGN

John W. Fisher[1]
William J. Wright[2]

Abstract

This paper examines the role that higher toughness of high performance steel has on the fatigue limit state. Test results on HPS-485W and HSLA-80 welded details are compared with the earlier studies used to develop the AASHTO fatigue design provisions. The test data demonstrates that the fatigue limit state remains the same. The high fracture resistance of HPS eliminates traditional cleavage (brittle) fracture. The available test results indicate that flange fracture only occurs when the net section reaches its tensile capacity. Fatigue enhancement of "traditional" low fatigue resistant details is demonstrated to be a feasible means of increasing fatigue strength. The ultrasonic impact treatment of Category C stiffeners was found to increase their resistance to Category B. Cover-plated Category E' was shown to improve to Category C. The process is easy to use and environmentally friendly. Other alternatives to minimize low fatigue resistant details by using new shapes such as corrugated or composite webs are examined.

[1] Joseph T. Stuart Professor, Civil and Environmental Engineering, Co-Director, ATLSS Research Center, Lehigh University.
[2] Research Structural Engineer, Turner-Fairbank Highway Research Center, FHWA.

Introduction

Since the late 1960's we have known that the yield strength of the steel has not had a beneficial effect on the fatigue resistance of welded details. This was recognized in the 1974 AASHTO fatigue design rules that introduced stress range as the governing limit state for all structural bridge steels (1).

Over the last two decades, steel producers worldwide have been installing advanced steel making practices including clean-steel technology, microalloying and thermomechanical control processing. These advances are leading to improvements in properties such as weldability, formability, fracture resistance, strength and ductility. Over the past decade low-carbon age-hardenable steels have gained increasing usage in shipbuilding, heavy vehicle manufacturing and offshore structures. It is clear from studies on new high-performance steels that the fundamental stress range-life relationships are not changed. The fatigue categories remain applicable to these steel materials, which include HPS-485W (70W) and HSLA 80 steel. These test results will be reviewed in this paper.

Related to fatigue resistance is the superior ductility and fracture performance of the high-performance steels. This has the beneficial effect of eliminating cleavage fracture (sudden brittle failure). As a result, reliability is enhanced as the net section can be relied on to resist the applied forces should any crack develop in the structure. Test results that demonstrate the superior resistance and tolerance of high-performance steel to fracture are also reviewed (2).

Studies on the use of high performance steel with higher yield strength suggest that fatigue is one of the barriers to effective use of high-performance steel, primarily due to fatigue-susceptible details. Therefore, fatigue susceptible welded details must be restricted or improved.

Among the characteristics most likely to improve the fatigue resistance of high-performance steel is the enhanced reliability due to ease of fabrication. Cracking of the base metal due to hydrogen is largely eliminated, and the ability to manufacture without preheat improves the working environment so that more reliable welds are produced.

Fatigue Resistance of Steels

The test data for welded plate girders without attachments are summarized in Figure 1 for the studies on C-Mn steels carried out in the 60's and 70's and for high performance HSLA 80 and HPS-485W steel girders in the 1990's (2, 3,4,5,6). Nominal stress range is plotted as a function of cyclic life for several levels of minimum stress and a wide range of yield points. These test results demonstrate that the Category B fatigue detail is not affected by the type of steel nor the ratio of minimum to maximum stress. Stress range alone is seen to define the fatigue resistance, as has been the case since adoption of the fatigue limit state S-N curves in 1974 (1).

Limited test data has also been developed on stiffeners welded to the web and flanges of HPS-485W plate girders. These test results are summarized in Figure 2 and compared with test results on large-scale girders tested in the 1970's fabricated from A36, A588 and A514 steel. All of these tests show direct correlation between these earlier tests and the more recent studies on the HPS-485W welded girders (4,6).

With the improvements in steel properties of HSLA-80 and HPS-485W steels, the weld metal has become an element of greater interest as higher weld strength metal is more difficult to weld. This has resulted in greater interest in the use of optimized groove welds. Much of the experimental work has focused on ultimate fracture behavior (tensile and shear strength). These studies have all confirmed that undermatched weld metal can develop the ultimate capacity of the connected plate material (8,9.10,11). Large-scale fatigue tests were carried out on optimized groove welds using E7018 shielded metal arc welding with HPS-485W steel (11). Those results are plotted in Figure 3 and compared with earlier tests on A36, A588 and A514 welded girder with matching weld metal. The test results show that the fatigue strength of flange transition groove welds is similar to the earlier tests on groove welded joints.

The data plotted in Figures 1, 2 and 3 show clearly that stress range is the only critical stress variable for all structural steels. The S-N curves shown in Figures 1 and 2 can be defined in log form by the relationship:

$$\log N = \log A - 3 \log S_r \tag{1}$$

In exponential form this becomes

$$N = A\, S_r^3 \tag{2}$$

Where N is the number of cycles to failure, S_r is the nominal stress range, A is a constant for a given detail, and 3 is the slope constant for these curves. The concept of fatigue design based on stress range alone was adopted by AASHTO in 1974. It was derived from newly developed knowledge described in Refs. 3 and 4. Small specimen data were discarded, as these specimens did not adequately capture residual stresses, geometry and weld discontinuities.

The stress range values given in the AAHSTO specifications in 1974 were originally derived from the 95% confidence limits for 95% survival. Rolled beams and base metal were Category A, welded girders were Category B and B', stiffeners and short 51 mm attachments were Category C, 102 mm attachments were Category D, and cover-plated beams were Category E and E'. The stress range cyclic life relationships are plotted in Figure 4 for each design category. The stress range approaches the crack growth threshold level for the various details and becomes a constant value or constant-amplitude fatigue limit (CAFL) which is indicated by the dashed horizontal lines in Figure 4. A comprehensive review of available test data resulted in all curves having the same constant slope 3 in Eq. 2 (7). These resistance curves now provide the fatigue strength design basis for the AASHTO, AREMA, AISC and AWS specifications. In addition, they also coincide with the Eurocode provisions. It is clear that these relationships remain applicable to HPS steels in this new century.

It should also be noted that the resulting exponential relationship between fatigue life and stress range described by Eq. 2 ($N = A\ S_r^{-3}$) is also compatible with the fracture mechanics of crack growth where the "Paris Law" governs the fatigue crack growth rate. The exponent of the Paris Law fit to steel crack growth rate data is also 3.0. Thus, both the S-N curves and the Paris Law give the same change in life with respect to changes in stress range.

Fracture Resistance

HPS-485W steel has much higher fracture toughness than the conventional grades of steel used for bridge construction. Figure 5 shows a Charpy-V-Notch (CVN) transition curve for HPS-485W compared to conventional grade 50W steel. The upper shelf toughness of the HPS steel is higher and the brittle–ductile transition occurs at a much lower temperature. This clearly shows that HPS-485W remains fully ductile at temperatures where conventional grade 50 steels begin to exhibit brittle behavior. This far exceeds the AASHTO toughness specification for zone III fracture critical applications (35 ft-lb. @ -10F).

The current AASHTO toughness requirements were designed to insure that purely brittle behavior (lower shelf) does not occur in bridges above their lowest anticipated service temperature. Most conventional steels ordered to these requirements are somewhere in the lower part of the transition region for low temperature service. The HPS-485W steels tested so far at the FHWA lab are all showing upper shelf behavior, even at the –60F extreme service temperature for zone III use. This is a fundamental change in behavior that results in much higher crack tolerance for the HPS steel.

Linear elastic fracture mechanics theory has been widely used to predict fracture for steels that exhibit brittle behavior. The critical stress intensity factor (K_{Ic}) is the toughness measure that is conventionally used to predict when fracture will occur from a fatigue crack. Barsom has shown that K_{Ic} can be calculated from CVN data using a two- step correlation procedure (13). Figure 6 shows a K_{Ic} transition curve for HPS-485W that calculated from CVN data using this correlation . This plot clearly shows that the transition to brittle behavior occurs far below the extreme zone III service temperature. Linear elastic theory, however, is only valid for brittle materials under plane strain conditions. K_{Ic} is not a valid parameter for predicting fracture for upper shelf materials. It can be seen that the current AASHTO toughness requirements based on linear elastic theory are not applicable to high toughness steels on the upper shelf. New ways to characterize fracture are required.

Research is underway to study different theories for predicting fracture in very ductile bridge steels. Test results at the FHWA laboratory are showing that net section yielding is a reasonable criteria to use to predict failure of HPS-485W (12). Full-scale I-girders are being subjected to cyclic loading to develop fatigue cracks. Figure 7 shows the test set-up at FHWA. Once fatigue cracks reach a given size, the beams are subjected to dynamic overloads to try to cause a fracture to occur from the fatigue crack. The beams are cooled to –30F prior to overload testing to simulate the most critical conditions that might cause fractures in zone II service. If

no failure occurs, the fatigue cracks are grown larger and the overload test repeated. Eventually the beams fail, either by yielding on the net section or by premature brittle fracture.

Figure 8 shows a profile of the cracked section of the tension flange of one of the test beams. The fatigue cracks were initiated from the weld toe of a Category C stiffener detail welded to the web. The size of the crack at each of the seven overload tests is indicated on the figure. Even with the large fatigue crack existing at overload number 7, the girder was able to resist the full design overload without fracture. Calculation of the theoretical bending stress on the cracked net section indicates that the stress in the tension flange was slightly higher than the yield strength of the material during the overload. About 50% of the tension flange was lost on the net section.

By contrast, similar full-scale fracture tests conducted at Lehigh University showed that conventional steels (A36, A572) were only capable of sustaining about 80% of the yield strength on the net section before brittle failure occurred (2). This shows that HPS-485W has a greater ability to resist fracture from fatigue cracks. Figure 11 shows the number of fatigue cycles that were required to grow the cracks shown in Figure 6. The first crack was detected around the lower 95% limit for fatigue Category C. About 1.5 million additional cycles were required to grow the crack through the flange to the size where net section yielding might occur.

Tolerance of cracks up to net-section yielding can offer several performance benefits for HPS bridges. Large crack tolerance increases the available time window for detecting and repairing fatigue cracks before the structure might become unsafe. A net section yield criteria provides an easy way for engineers to assess the significance of cracks once they are found. Preliminary research is also indicating that it might be possible to relax the rules currently penalizing non-redundant structures.

Fatigue Enhancement

Weld Toe Improvement Techniques

Weld toe improvement techniques, including grinding, air hammer peening, gas tungsten arc (GTA) remelting, ultrasonic-impact treatment (UIT), and welding consumables with improved weld toe characteristics, have been developed and tested worldwide. Several techniques are available to improve fatigue strength of welded details. Both laboratory studies and field applications of weld toe treatments have been made using air hammer peening, GTA remelting, and UIT.

Various methods for repairing or retrofitting cracked bridge details were studied by Fisher, et al. for NCHRP Report 206 (14). Techniques studied included grinding, air hammer peening, and GTA remelting the fillet weld toe. These methods were applied to the ends of coverplates with known fatigue cracks in full-scale welded girders. After these laboratory tests, cracked beams of the Yellow Mill Pond Bridge on I-95 were retrofitted at the bridge site by either air hammer peening or GTA remelting on two spans. The retrofits were successful, and no

detectable fatigue cracks were observed at the treated coverplate ends after 20 years of service on I-95, when the girders were removed from the bridge and examined in the laboratory.

The Category E' coverplated beams that were removed from the I-95 Yellow Mill Pond Bridge in 1997 had been retrofitted in 1976 by either air hammer peening or gas tungsten arc (GTA) remelting. All details had small fatigue cracks at the time of retrofit. The beams were tested at a stress range of 69 MPa (10 ksi). Cracks developed from the root of the transverse end weld and propagated through the weld throat. The fatigue resistance of the treated weld toe details improved to Category C except for one GTA-remelted detail, which exceeded Category D (6).

The UIT method of postweld improvement, developed by E. S. Statnikov at the Northern Scientific and Technological Fund in Severodvinsk, Russia (15), introduces ultrasonic frequency impacts on the weld toe surface, which produces compression residual stresses in the treated areas. In addition, a visible improvement can be observed in the weld toe profile. The UIT tool is shown in Figure 9. Details of the tool and electronic control box remain proprieitary. The UIT tool is easy to handle during application compared with conventional air hammer peening, and its use was easily taught. The frequency of the head movement is indicated to be close to 27,000 Hz. The noise is negligible compared with that from air hammer peening. Several kinds of heads and tips are available and can be chosen on the basis of the surface condition of the weld details. The UIT tool is connected to a control box, which controls the intensity of UIT energy and frequency.

Three plate girder specimens with coverplated and transverse stiffener details were produced by a local bridge fabricator. The girder geometry is shown in Figure 10. Each girder was 6.7 m (22 ft.) long and was tested on a 6.4 m (21 ft.) span under two point loading. The material of flange plates and web plates was HPS-485 W steel [F_y = 485 MPa (70 ksi), F_u = 620 to 760 MPa (90 to 110 ksi) and that of the coverplates and the stiffeners was ASTM A588 Grade 50 weathering steel.

The 25 mm (1 in.) coverplate and the transverse stiffener details are classified as Category E' and C, respectively. Altogether, eleven of the weld details on these specimens were treated by UIT.

The test data for as-welded and UIT-details are plotted in Figure 11, along with the results of previous research (7). The data for untreated details are consistent with previous research, with data from transverse stiffeners falling above the AASHTO Category C fatigue resistance curve and data from the coverplates falling above the Category E' curve. Conversely, data from all the UIT details provided significantly longer life and were shifted to the right compared with the untreated details. Only one UIT detail developed a fatigue crack. Data from the transverse stiffener details given UIT fall between the Category A curve and the Category B curve, whereas those of the untreated details fall near the Category C curve. Similarly, data from the coverplate details given UIT fall along the Category C curve, and those of the untreated details fall along the Category E' curve at a lower stress range level.

Other Alternatives

An attractive means of eliminating Category C stiffeners from plate girder webs is the possibility to use steel web plates with corrugations. There has been extensive use of corrugated web girders in building applications but only a few applications in bridge structures with most using two concrete flanges (16). Although a few fatigue tests have been carried out on very thin (2 to 4 mm) corrugated web girder (17,18), these tests are not representative of large scale girders as is well known from traditional fatigue studies (3,7).

A demonstration project now underway at Lehigh University will test six, 6 mm thick corrugated web girder test specimens that are 7 m (23 ft.) long with full-scale cross-sections (1.2 m) deep, as illustrated in Figure 12. Three specimens are being tested at each of two stress range levels (140 MPa and 105 MPa). It is anticipated that the fatigue resistance will approach Category B.

Other possibilities include the use of composite sandwich webs. This too would avoid the use of transverse stiffeners which limits the fatigue resistance to Category C. Otherwise weld improvement treatments such as air-hammer peening or ultrasonic impact treatment (UIT) must be used to improve the fatigue strength of the details.

Summary

Recent tests at the ATLSS Center and at the FHWA Turner-Fairbanks Laboratory have confirmed that the fatigue strength of high performance steel weld details is the same as that of conventional steel weld details. Hence, the AASHTO Specification fatigue limit state is appropriate for HPS steel bridge structures. Fatigue-susceptible details must be avoided or appropriately enhanced. Weld detail improvement treatments such as air hammer peening or ultrasonic impact treatment (UIT) have been shown to improve the fatigue strength of welded details. Large scale tests carried out on HPS-485W and other tests currently underway have demonstrated that Category E' coverplated beams can achieve Category C and transverse stiffeners and connection plates can achieve Category B fatigue resistance.

Other alternatives to enhancing fatigue resistance require developing new cross-sections that can avoid using welded transverse stiffeners on girder webs. An HPS demonstration project now underway in Pennsylvania is assessing the fatigue resistance of full-scale corrugated web plate girders.

Acknowledgements

This paper draws upon a number of studies on the fatigue and fracture resistance of HPS steels. These studies are supported by the Federal Highway Administration, the Pennsylvania Department of Transportation, the Pennsylvania Infrastructure Alliance, and the American Iron and Steel Institute.

References

1. Fisher, J. W., (1977), *Bridge Fatigue Guide: Design and Details*, American Institute of Steel Construction, AISC Publication No. T112-11/77, 1977.

2. Roberts, R., Fisher, J. W., Irwin, G. R., Boyer, K. D., Hausammann, H., Krishna, G. V., Morf, V. and Slockbower, R. E., (1977), "Determination of Tolerable Flaw Sizes in Full Size Welded Bridge Details," Report No. FHWA-RD-77-170, Federal Highway Administration, Washington, D.C., 1977.

3. Fisher, J. W., Frank, K. H., Hirt, M. A. and McNamee, B. M., (1970), *Effect of Weldments on the Fatigue Strength of Steel Beams,* National Cooperative Highway Research Program Report 102, Highway Research Board, Washington, D.C., 1970.

4. Fisher, J. W., Albrecht, P. A., Yen, B. T., Klingerman, D. J. and McNamee, B. M., (1974), *Fatigue Strength of Steel Beams with Transverse Stiffeners and Attachments*, National Cooperative Highway Research Program, Report 147, Highway Research Board, Washington, D.C., 1974.

5. Dexter, R. J., Fisher, J. W. and Beach, J. E., (1993), "Offshore Mechanics and Artic Engineering Conference, June 1993, Glasgow, United Kingdom.

6. Takamori, H. and Fisher, J. W., (2000), "Tests of Large Girders Treated to Enhance Fatigue Strength," *Transportation Research Record 1696*, Transportation Research Board, Washington, D.C., 2000.

7. Keating, P. and Fisher, J. W., (1986), "Evaluation of Fatigue Tests and Design Criteria on Welded Details," *NCHRP Report 286*, Transportation Research Board, Washington, D.C., September 1986.

8. Sato, K. and Toyoda, M., (1975), "Joint Strength of Heavy Plates and Lower Strength Weld Metal," *Welding Journal Research Supplement*, Vol. 54, No. 9, September, 1975.

9. Dexter, R. J. and Ferrell, M., (1995) "Effects of Weld Metal Strength and Defects on the Ductility of HSLA-100 Plates," Proceedings 14th International Conference on Offshore Mechanics and Arctic Engineering (OMAE), Copenhagen, Denmark, ASME, Vol. III, *Materials Engineering*, 1995.

10. Umekuni, A. and Masubuchi, K..(1997), "Usefulness of Undermatched Welds for High-Strength Steels," *Welding Journal Research Supplement*, Vol. 76, No. 7, July 1997.

11. Kaufmann, E. J. and Pense, A. W., (2000), *Innovative HPS-70W Ford City Bridge Demonstration Project: Improved Weldability Using Optimized Weld Metal Strength,"* ATLSS Final Report to Pennsylvania Department of Transportation, April 2000.

12. Wright, W. J. (2000), Personal Communication on Test Results (see Figure 11).

13. Barsom, J. M. and Rolfe, S. T., (1987), "Fracture and Fatigue Control in Structures," 2nd Edition, Prentice Hall, Englewood Cliffs, NJ, 1987.

14. Fisher, J. W., Hausammann, H., Sullivan, M. D. and Pense, A. W., (1979), "Detection and Repair of Fatigue Damage in Welded Highway Bridges," *NCHRP Report 206*, TRB; National Research Council, Washington, D.C., 1979.

15. Statnikov, E. S., (1997), "Applications of Operational Ultrasonic Impact Treatment (UIT) Technologies in Production of Welded Joints," *IIW/IIS Document X11-1667-97*, International Institute of Welding, 1997.

16. Brozzetti, J., (1991), "Steel Bridge Construction in France," Proceedings, National Symposium on Steel Bridge Construction, St. Louis, MO, FHWA, Washington, D.C., 1991.

17. Korashy, M. and Varga, J., (1979), "Comparative Investigation of Fatigue Strength of Beams with Web Plate Strength of Beams with Web Plate Stiffened in the Traditional Way and by Corrugations," *Acta Technica*, Academiae Scientarum Hunggaricae, 1979.

18. Elgaaly, M., Seshadri, A., Rodriquez, R. and Ibrahim, S., (2000), "Bridge Girders with Corrugated Webs," *Transportation Research Record*, 1696, Paper 5B0022, 2000.

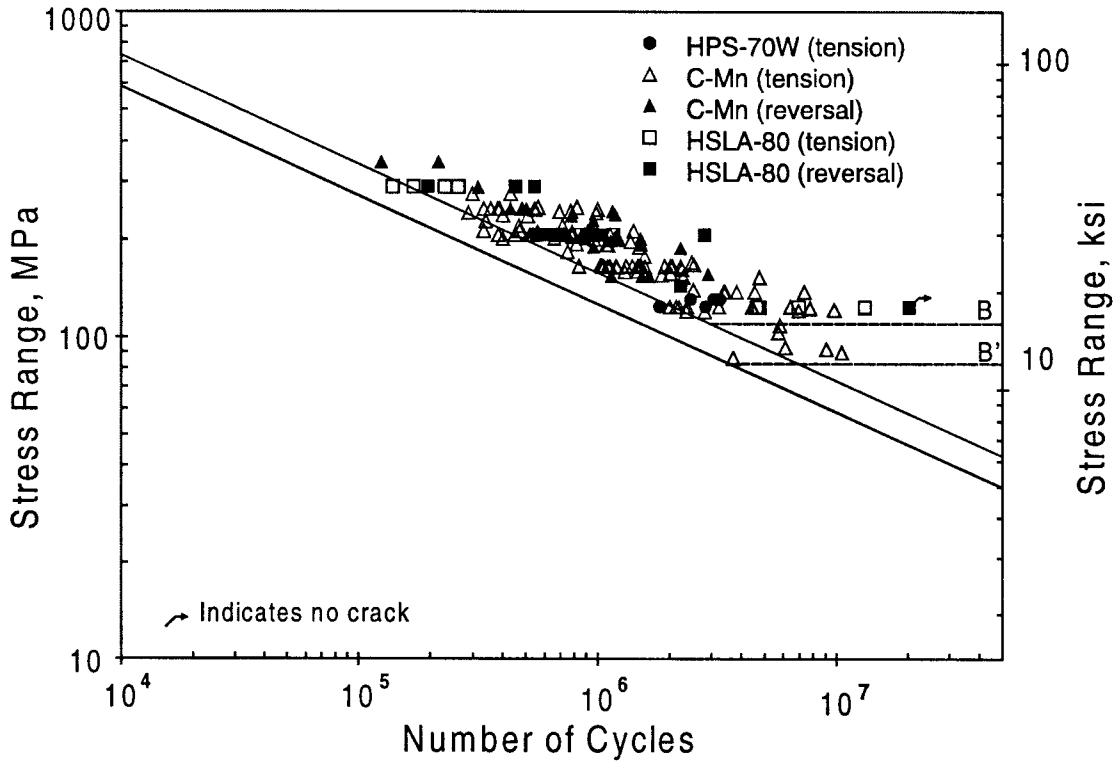

Figure 1 Fatigue Resistance of web flange fillet welds (Category B)

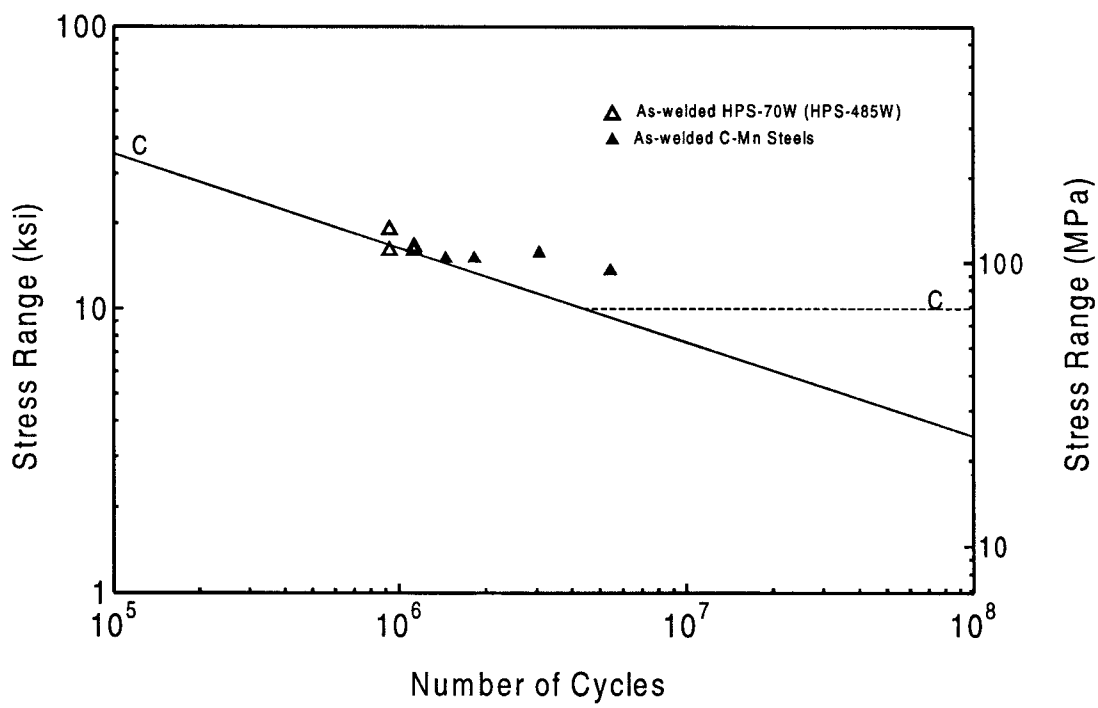

Figure 2 Fatigue Resistance of transverse stiffeners on welded girders (Category C)

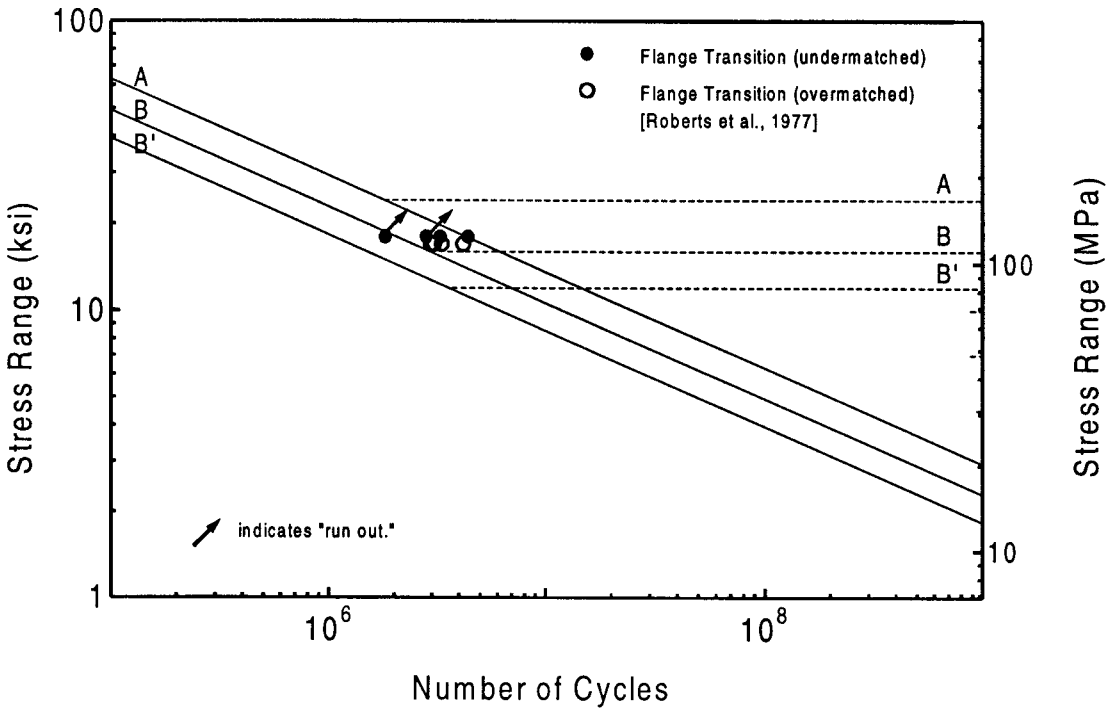

Figure 3 Groove Welds with undermatched and overmatched weld metal (Category B)

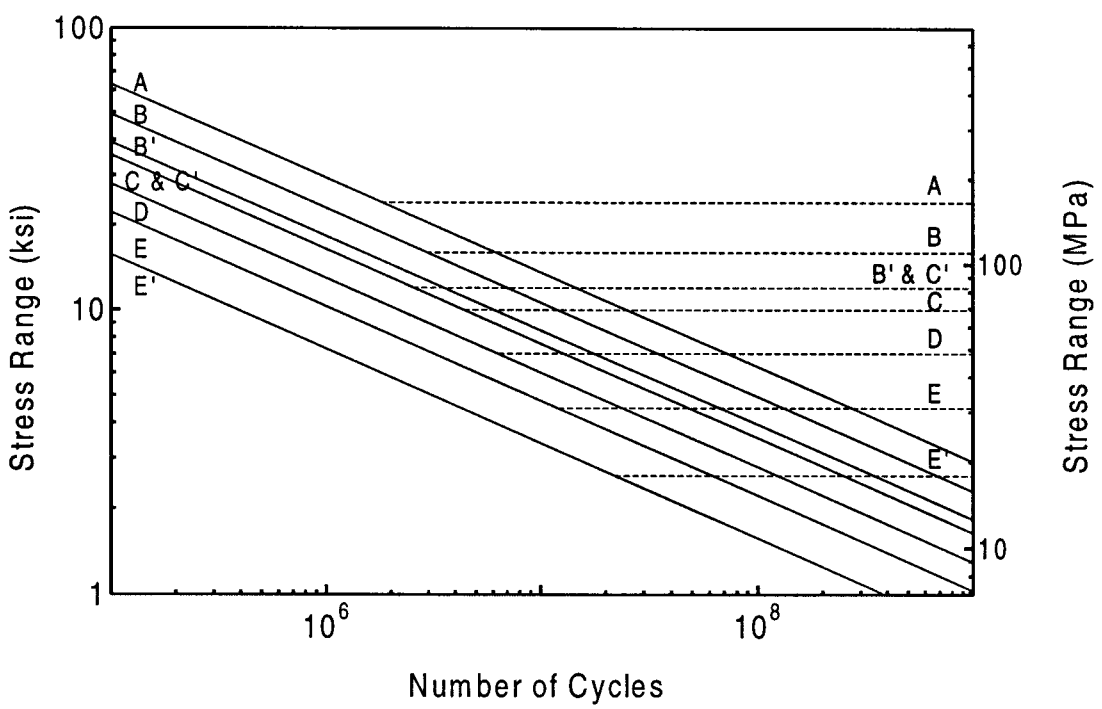

Figure 4 Design fatigue resistance curves for Categories A to E'

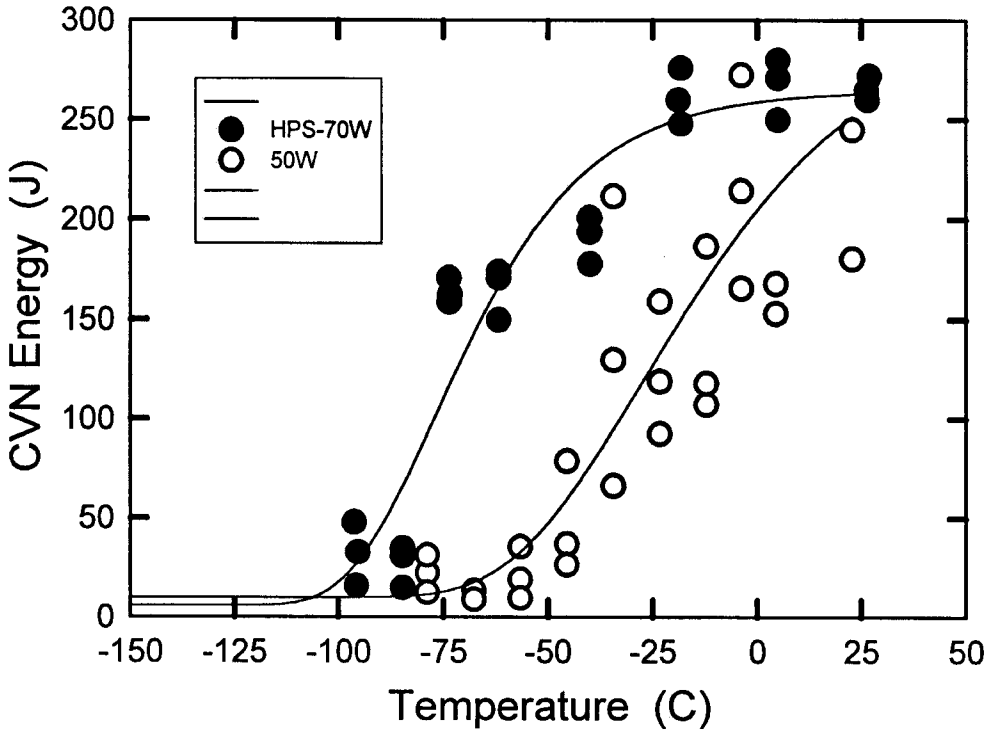

Figure 5 CVN transition curve for HPS-485W compared to A588 Steel

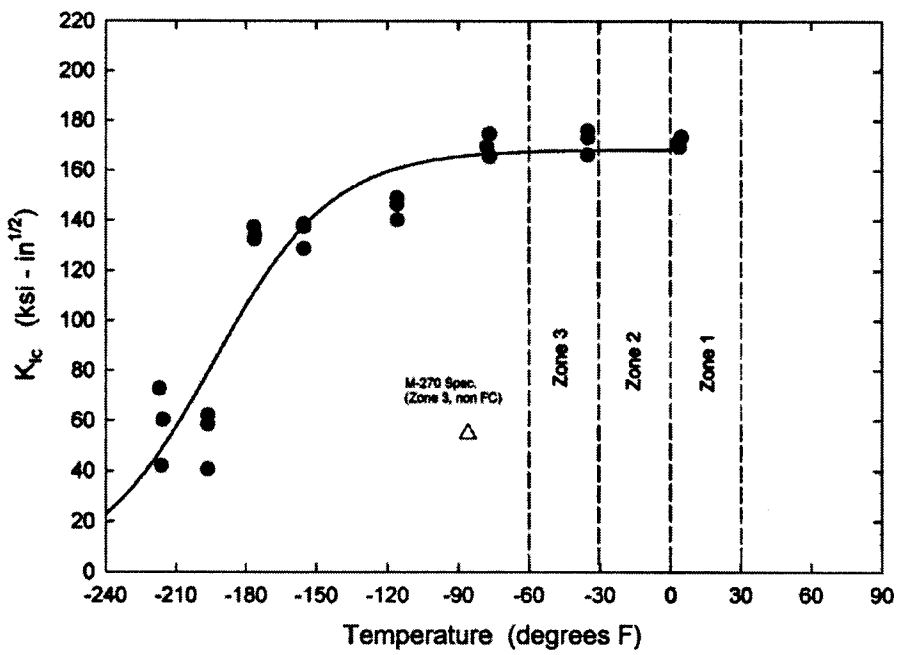

Figure 6 KIc transition curve for HPS-485W calculated from CVN data

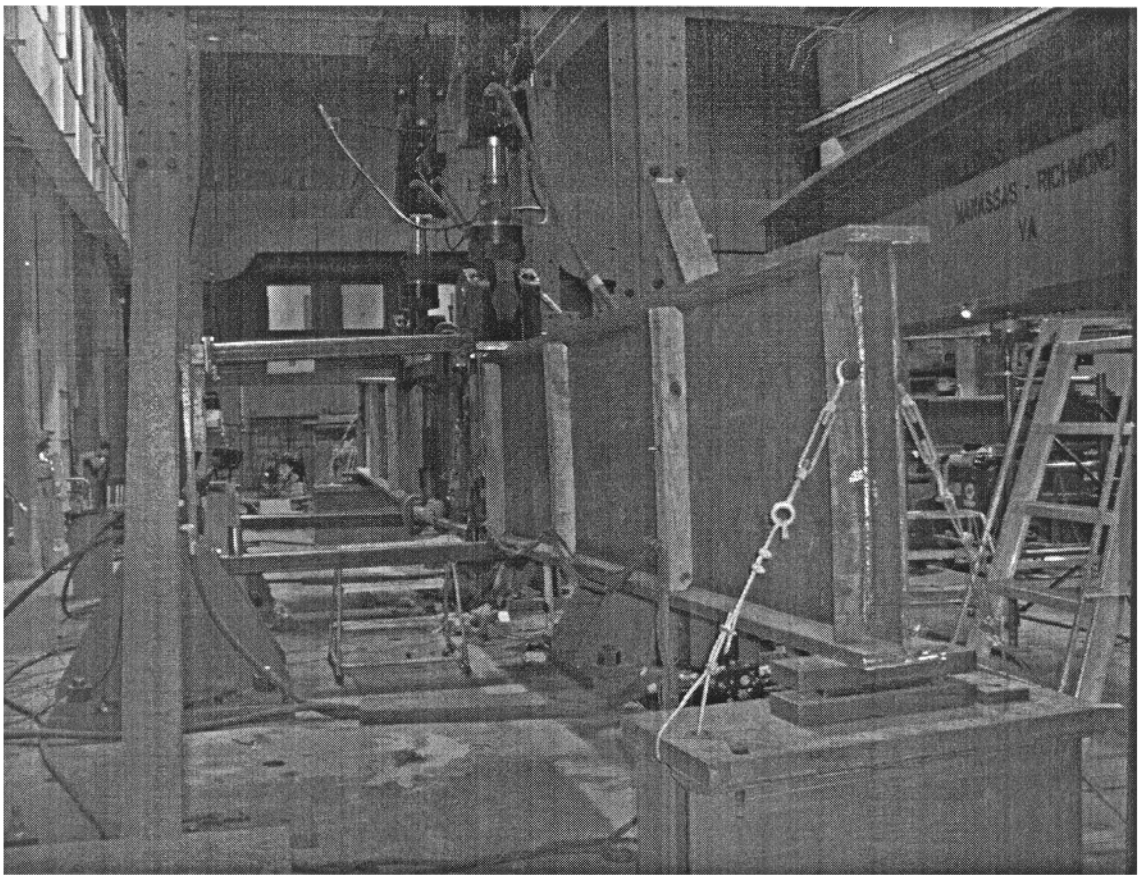

Figure 7 Beam fractured after the yield strength was exceeded on the fatigue cracked net section.

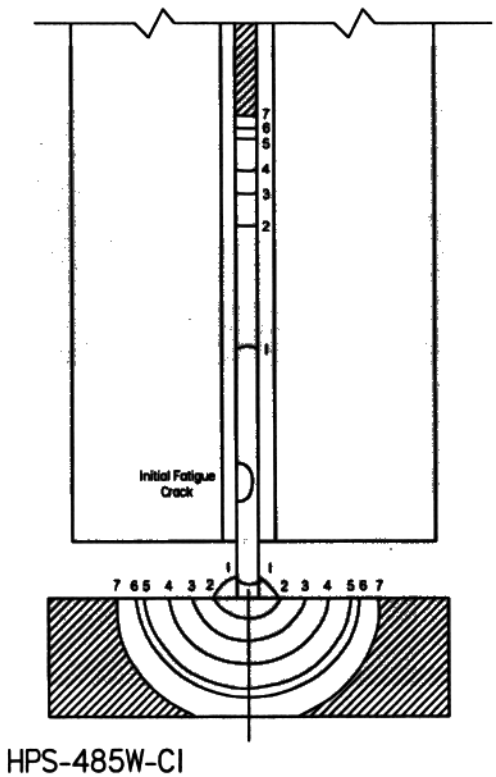

Figure 8 Fatigue crack history for full-scale girder test at FHWA (S_r=93Mpa)

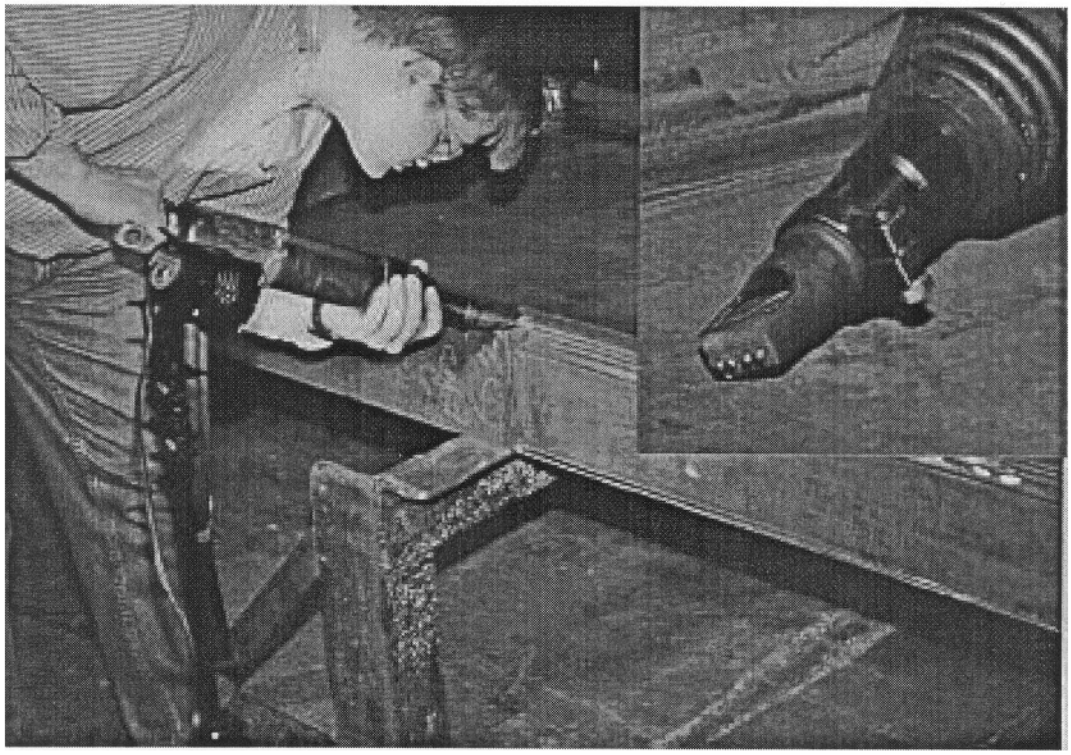

Figure 9 Ultrasonic Impact Treatment (UIT) of HPS-485W girder

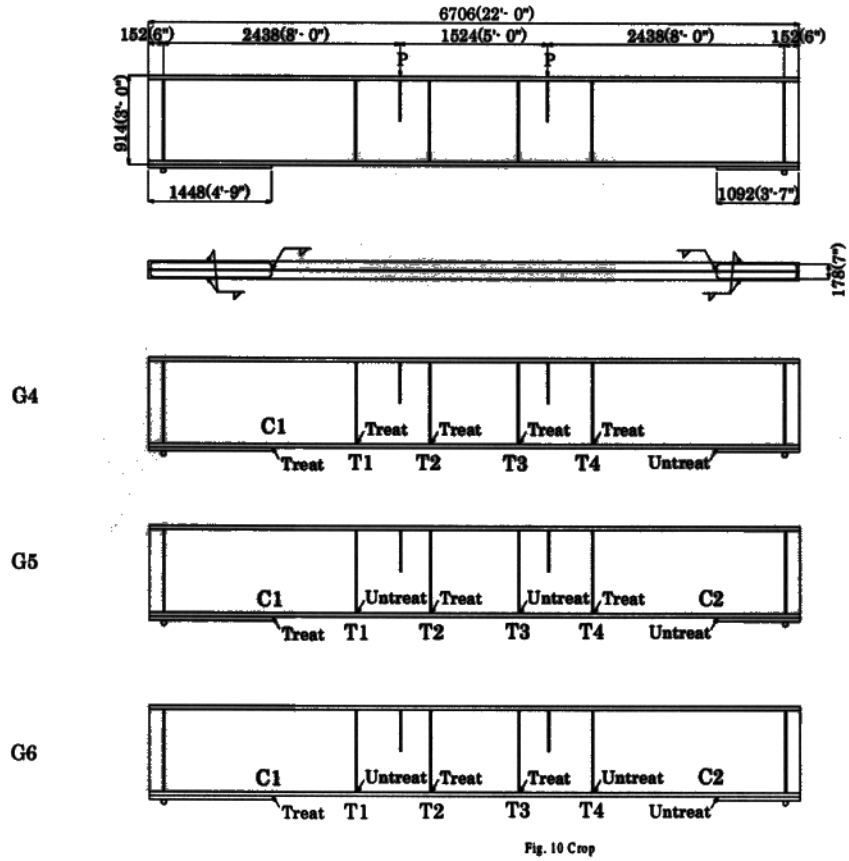

Figure 10 HPS-485W test specimens for treated (UIT) and untreated weld details

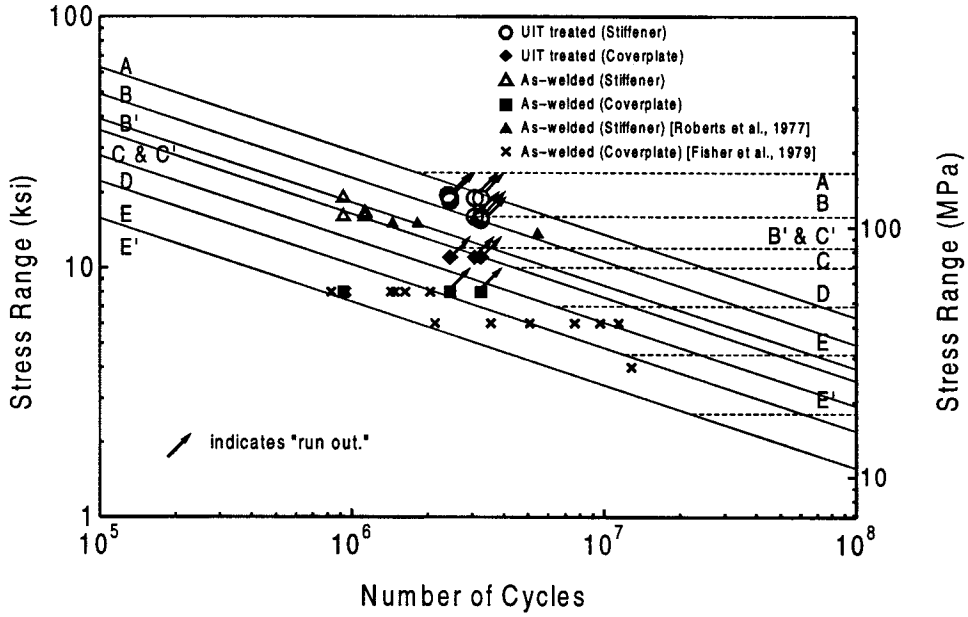

Figure 11 Comparison of UIT enhanced weld details with untreated details

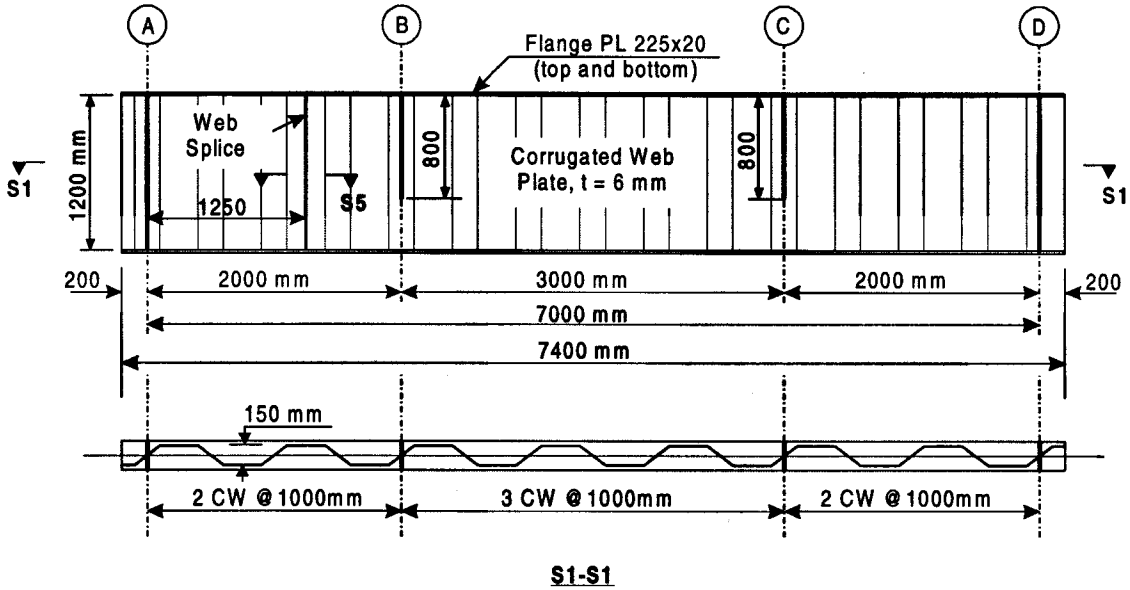

Figure 12 Schematic of Corrugated Web Girders for Fatigue Testing