Fatigue Behavior of Weathered Steel Components

J.M. BARSOM

ABSTRACT

The effect of weathering on the fatigue behavior of fabricated weathering-steel components used for bare applications in bridges and other structures is discussed. The fatigue behavior of weathered weathering-steel and unweathered steel structural details is compared, and the applicability of AASHTO fatigue-design curves to predict their behavior is discussed. The data and discussion reveal that surface roughness of steels caused by weathering corresponds to localized stress (strain) raisers on the surface that may decrease the fatigue life of weathered weathering-steel components. Consequently, the effect of weathering on fatigue life is more pronounced for category A details. However, because the surface imperfections that correspond to the various AASHTO fatigue-design curves are more severe than those generated by weathering, the current AASHTO fatigue-design curves should be equally applicable to predict the fatigue behavior of weather and unweathered bridge-steel components.

Unprotected structural steels are oxidized by aqueous environments. This corrosion process occurs on the exposed surfaces and transforms the steel surface into corrosion products. Because of localized variations in the electrochemical reactions during the corrosion process and in the transport of the environment through the corrosion products, the oxidation process proceeds at slightly different rates in neighboring regions. This localized variation results in roughening of initially smooth surfaces that are exposed to the environment. Surface roughness corresponds to localized stress (strain) raisers on the surface that may decrease the fatigue life of weathered components.

Weathering steels (such as ASTM A588) subjected to full-immersion conditions in water corrode at the same rate as carbon steels. However, unlike carbon steels, weathering steels subjected to wet and dry weathering cycles form a highly adhering oxide layer that, with time, significantly retards further oxidation. During the time necessary to develop a protective oxide layer, the underlying surface of the steel is roughened by the wet and dry weathering process.

The effect of weathering on the fatigue behavior of fabricated weathering-steel components used for bare applications in bridges and other structures is discussed. The fatigue behavior of unweathered fabricated-steel details and the principles used in the development of the AASHTO fatigue-design curves are described. Then the effects of weathering, if any, on the fatigue behavior of fabricated-steel details are presented, and the applicability of the AASHTO fatigue-design curves to predict the behavior of weathered fabricated-steel components is discussed.

GENERAL FATIGUE BEHAVIOR OF STRUCTURAL COMPONENTS

The fatigue life of any structural component can be divided into an initiation life and a propagation life. The existence of stress raisers (such as changes in geometry, notches, or welding imperfections) minimize or may eliminate the initiation life. Stress raisers can be classified as geometry related (such as changes in cross section) or imperfections (such as gouges and weld imperfections). Fatigue cracks always initiate at the geometrical or imperfection stress raiser that causes the highest localized stress intensification. Thus, under identical test conditions, a machined and polished specimen would have a longer fatigue life than the same specimen that has mill surfaces that, in turn, would have a longer fatigue life than the same specimen that contains a severe gouge.

Once a crack is initiated the remaining life is governed by the stress range and the crack size, such that the fatigue life decreases as the magnitude of each of these parameters increases.

It can be concluded from the preceding discussion that, among other things, stress raisers decrease the fatigue life of structural components, and that the shortest life of otherwise identical details is obtained for the component that contains the most severe stress raiser.

FATIGUE BEHAVIOR OF UNWEATHERED STRUCTURAL-STEEL COMPONENTS

To evaluate the significance of weathering on the fatigue behavior of structural-steel components, the fatigue behavior of unweathered details and some of the principles used in the development of the AASHTO fatigue-design curves need to be understood. The behavior of unweathered details can then be used as reference to establish the significance, if any, of weathering on the fatigue behavior of structural-steel components.

AASHTO Fatigue-Design Curves

The current AASHTO fatigue-design specifications are based on experimental curves that relate the fatigue life to failure (number of cycles), N, of unwelded and welded details to the total (tension plus compression) applied nominal stress range, $\Delta\sigma$ (1). A large number of tests for a given detail were conducted and compared with other available data to generate statistically significant stress-range versus fatigue-life relationships. The design curves represent the 95 percent confidence limit for 95 percent survival of all available data for a given detail.

The extensive fatigue data that have been obtained by testing bridge details have been used to establish allowable stress ranges for various categories of details (Figure 1). Each category represents structural details that have approximately equivalent fatigue strengths. For example, all welded attachments that have a length (L) in the direction of stress equal to or less than 2 in. are considered to have equivalent fatigue strength. In reality, under identical fabrication and geometrical conditions, a 2-in.-long attachment results in a higher stress concentration than a shorter attachment and, therefore, would have a shorter fatigue life. Because the curve for each category corresponds to the 95 percent confidence limit for 95

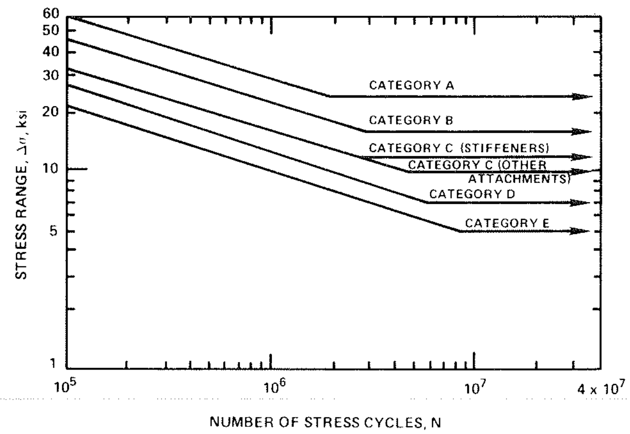

FIGURE 1 AASHTO design stress range curves for categories A-E.

percent survival of all the details in a given category, the fatigue-design curves correspond to approximately the shortest lives obtained for details in each category and are, therefore, governed by the details in that category that have the most severe geometrical or weld-stress concentration.

The existence of gouges and weld-imperfection stress raisers in a structural detail of a given geometry decrease the fatigue life of the detail. Consequently, significant variability (scatter) in fatigue-life data can be obtained by testing many details of identical geometry, but which contain different size imperfections. This variability in the data is apparent in the data base used to establish the AASHTO fatigue categories. For example, the longest life obtained for a category C detail (stiffener) that was tested at a stress range ($\Delta\sigma$) of about 25 ksi was about 4 times longer than the same detail that exhibited the shortest life. The difference in fatigue life for these two specimens was caused primarily by the difference in the size of the initial imperfections that existed in the specimens. The one that had the shortest life contained the most severe stress raiser. Because the shortest life is obtained for the specimen that contained the most severe imperfection, the AASHTO fatigue curve for each category is governed by the specimens that contained the most severe imperfection. Thus the AASHTO fatigue curves represent the 95 percent confidence limit for 95 percent survival of all the details in a given category and are governed primarily by the details in a given category that have the most severe geometrical discontinuities, imperfections, or both.

Imperfections in Structural Components

The AASHTO fatigue categories encompass base metal (category A) as well as weldments (categories B-F). Consequently, the imperfections that are the origin of fatigue cracks can be divided into three categories: (a) imperfections in base metal, (b) imperfections embedded in weld metal, and (c) imperfections at a weld termination or weld toe. The following is a brief characterization of the imperfections that are of primary significance to each of the AASHTO categories.

Category A

The AASHTO fatigue-design curve for category A determines the allowable stress ranges for base metal with as-rolled or cleaned mill surfaces and flame-cut edges that have a surface roughness value no greater than 1,000 microinches (μin.) R_a (arithmetic average roughness), as defined by the American National Standards Institute (ANSI) (2, p. 4).

The available data (3-8) indicated that "rolled beams provide the least severe flaw condition for structural elements and can yield extremely long lives at high stress-range levels; however, a large discontinuity in the surface or at the flange tip can reduce the fatigue life of the beam substantially" (4, p. 11). Thus a few rolled beams that contained large discontinuities, which are not permitted by the current specifications, yielded fatigue lives equivalent to the mean life for welded beams (category B).

Examination of welded-beam test results obtained at Lehigh University (3,4) indicated that, in general, the most severe imperfections resided in the fillet welds rather than at the flame-cut edges. The fatigue performance of good-quality [arithmetic average roughness (R_a) of 1,000 μin. or less] flame-cut edges was closer to that for rolled beams than for welded beams. Consequently, flame-cut edges that have an ANSI roughness of 1,000 μin. R_a or less are included in category A.

The notches introduced by good-quality flame cutting (R_a of 1,000 μin. or less) that caused failure were more severe and sharper than the notches introduced in rolled beams and plate surfaces by the rolling operation (4, p. 10). Because the shortest lives were obtained for the specimens that contained the most severe imperfections, the AASHTO fatigue-design curve for category A included those specimens that contained the most severe imperfections generated by good-quality flame cutting (R_a roughness $\leq$1,000 μin.).

Category B

The welded details that are encompassed in AASHTO category B are primarily fillet welds, groove welds with welds ground flush, and groove welds in transition joints that have generous slopes (no steeper than 1 to 2.5) and radii ($\geq$24 in.). These ground slopes and radii were selected to minimize the effects of the geometrical stress concentrations and thus force the fatigue cracks to initiate from subsurface weld imperfections.

Fatigue cracks in category B weldments originated at porosity, lack of fusion, weld repair, tack weld, stop-start position in the longitudinal flange-to-web fillet weld, or trapped slag (9). The majority of cracks initiated and propagated as subsurface cracks until they intersected the fillet-weld surface. The few cracks that started from the flange tip contained notches that were "visually apparent and more severe than the regular flange roughness caused by the flame-cutting procedure" (3, p. 22). Beams that failed by this mode yielded shorter lives than those that failed from weld imperfections. Despite the severe damage to the flange edges, the short lives for these beams exceeded the design life provided by the category B fatigue-design curve.

Because AASHTO specifications require the removal of severe gouges and notches from flanges, the most likely imperfection that would cause fatigue failure of plain welded beams is an internal weld imperfection.

Categories C-E

The majority of fatigue cracks in bridge girders initiate at a weld toe or at a weld termination near a stiffener, or other attachments such as a gusset plate or end of a cover plate. These are regions of high stress concentration and high residual stresses that may contain small weld imperfections such as slag intrusion (9,10). Moreover, because the surface of the deposited weld metal is invariably rippled, the toe angle between the weld metal and the base metal can vary significantly at neighboring points along the weld toe, thereby resulting in variations in the stress concentration. For a cover plate with longitudinal fillet welds, the fatigue crack initiates at the termination of the weld. For a cover plate with transverse fillet welds, multiple fatigue cracks initiate at the toe of the weld.

Because fatigue cracks in categories C-E initiate from similar weld imperfections at weld toes and weld terminations, the decrease in fatigue life from category C to category E is related primarily to an increase in the severity of the geometrical stress raiser at the toe or termination of the weld. This severity is dependent on geometrical factors as well as on the quality of fabrication.

EFFECT OF WEATHERING ON THE FATIGUE PERFORMANCE OF WEATHERING-STEEL STRUCTURAL DETAILS

Surface roughening of steels that is caused by weathering corresponds to localized stress (strain) raisers on the surface that may decrease the fatigue life of weathered components. The change in fatigue life of a component caused by weathering depends primarily on the magnitude of the most severe stress raiser induced by weathering as compared with the magnitude of the most severe stress raiser residing in the component before weathering. Thus the decrease in fatigue life caused by weathering should be most pronounced for machined and polished components and negligible, if not beneficial, for components that contain severe surface notches or imperfections from other sources. Thus the magnitude of the effect of weathering on the mean fatigue life is strongly dependent on the intitial condition of the unweathered components, such that the mean fatigue-life curves for components with the smallest surface and subsurface imperfections exhibit the largest effects. Consequently, the effects of weathering on the AASHTO fatigue-design curves rather than on the mean fatigue-life curves should be investigated. Also, the correspondence between the most severe stress raiser encompassed by each AASHTO fatigue category and those induced by weathering need to be considered.

Surface Roughness Caused by Weathering

The arithmetic average surface roughness (R_a) for weathering-steel samples that have been weathered for up to 11 years was measured in accordance with ANSI procedures by using a standard roughness sampling length (cutoff) value of 0.03 in. The specimens weathered for 11 years had as-received mill surfaces and were exposed in a moderate marine environment. Specimens that were weathered for 6 years had either as-received mill-scaled surfaces or blast-cleaned surfaces and were weathered in a semi-industrial environment. All specimens were cleaned as recommended by ANSI/ASTM G1-72 in a 1 to 2 percent solution of sodium hydride in molten sodium hydroxide at 700°F.

Figure 2 shows the R_a values as a function of exposure time. The data indicate that the R_a values for all specimens and test conditions reach a constant maximum value of about 600 μin. between 2 and 3 years. Moreover, the peaks-per-inch count (of peaks greater than 50 μin., peak to valley) as a function of time, which is another surface roughness parameter, for all specimens and test conditions (Figure 2) also indicates that the surface roughness asperity density reaches a constant minimum value of about 100 peaks per inch between 2 and 3 years.

Because R_a, and therefore the depth of the surface pits, reaches a constant maximum value, and the peaks per inch reach a constant minimum value, the surface roughness induced by weathering corresponds to gentle craters rather than to sharp notches.

Effect of Weathering on AASHTO Fatigue Categories

The AASHTO fatigue-design curves represent the 95 percent confidence limit for 95 percent survival of all the details in a given category and, therefore, correspond to the fatigue behavior of the components that have the most severe geometry, imperfections, or both in that category. Therefore, most of the details in a given category should have longer fatigue lives than predicted from the design curve for that category. For example, a butt-welded component with the weld ground flush (category B) can be fabricated with minimum or no imperfections such that its fatigue life for a given stress range is as good as the life for the as-received plate or rolled beam (category A). The fatigue cracks for such a category B detail would more likely initiate from surface rather than from subsurface imperfections. Weathering, which is a surface phenomenon, would be expected to decrease the fatigue life of such a detail more than for a similar category B detail that contains a severe subsurface imperfection. Consequently, the effect of weathering on the fatigue

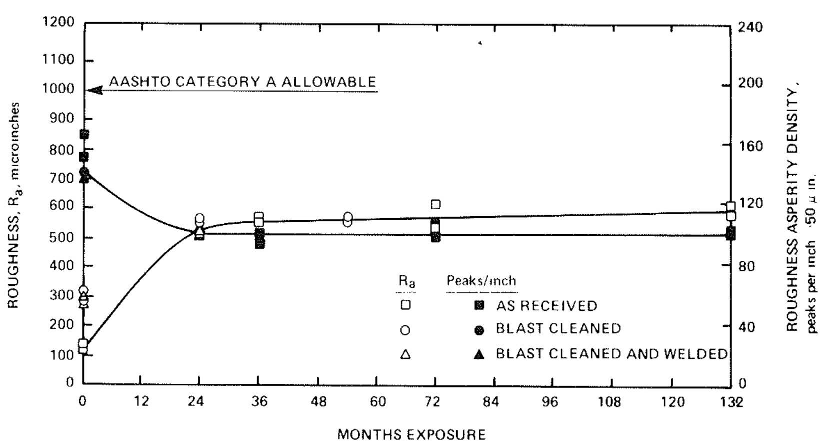

FIGURE 2 Surface roughness versus exposure time for weathering steels.

life of steel components must be considered in reference with its effect on the AASHTO fatigue curves rather than on the curves that correspond to the mean fatigue lives. The AASHTO fatigue curves represent the fatigue behavior of components that contain the most severe imperfections allowed for a given category, rather than the behavior of components that contain the least severe imperfections and therefore would exhibit superior fatigue behavior.

Category A

Fatigue tests conducted on unweathered category A type specimens indicated that the failure-causing notches introduced by good-quality flame cutting ($R_a \leq$ 1,000 μin.) were more severe and sharper than the notches introduced in rolled-beam and plate surfaces by the rolling operation (4, p. 10). Consequently, the AASHTO category A fatigue curve corresponds to the fatigue behavior of components that contain notches that have a severity equivalent to at least the severity of a flame-cut edge with an R_a of 1,000 μin. Figure 2 shows that the ANSI roughness of weathered surfaces reaches a maximum of 600 μin., which is lower than the 1,000 μin. allowed for flame-cut edges. Therefore, the fatigue life of weathered weathering-steel surfaces should be longer than the life of flame-cut edges with 1,000 μin. roughness and, at a given stress range, longer than predicted by the AASHTO category A fatigue-design curve. This observation is supported by experimental data obtained at U.S. Steel Research for specimens with as-received mill-scaled surfaces and with blast-cleaned surfaces (Figure 3). (Note that these data are from an unpublished report by G.T. Blake, "Fatigue Tests of A588 Steel at Different Exposure Times During Six-Year Weathering," July 1982.)

Because weathering is a surface phenomenon that induces surface stress raisers, and because fatigue cracks for category A initiate at surface imperfections rather than from weld imperfections or from geometrical stress raisers as for categories B-E,

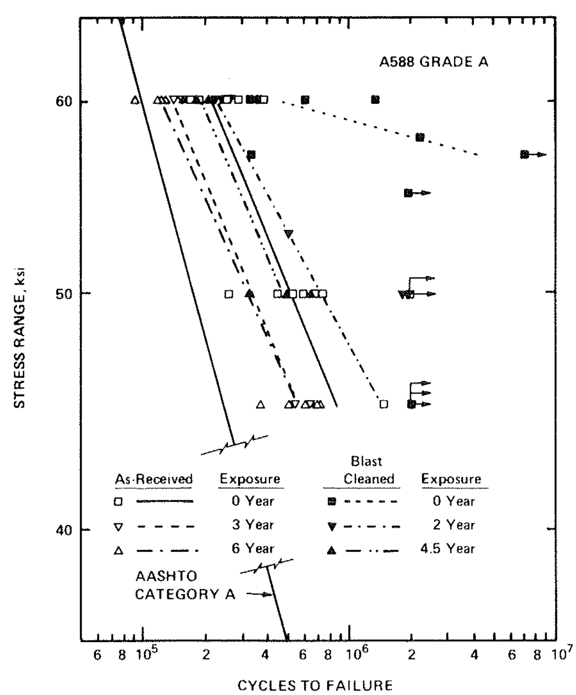

FIGURE 3 Fatigue data for as-received and blast-cleaned specimens.

the effect of weathering on the fatigue life of weathering-steel components should be most pronounced for category A type components than for any other component.

The fatigue behavior of weathered steel specimens was investigated by several Japanese organizations in a round-robin program reported by Kunihiro et al. (11). The investigation included (a) carbon steels and weathering steels, (b) two strength levels for each type of steel, (c) base metal specimens and butt-weld specimens that were ground flush, (d) three exposure sites that had different atmospheric severity, and (e) three exposure-time durations: 0, 2, and 4 years.

The following discussion presents an analysis of all the Japanese data for base metal specimens of the two grades of weathering steels that were exposed at three sites for 0, 2, and 4 years. These combined data have been reported by Albrecht (12) (Figures 4 and 5). These data indicate that, for stress ranges larger than about 35 ksi, some of the fatigue lives for weathered and, to a lesser extent, unweathered specimens were smaller than would be predicted by the AASHTO category A fatigue-design curve.

The data reported by Kunihiro et al. (11) were presented in a tabular form with comments that usually defined the location for the fatigue-crack initiation site. These comments indicate that the failure of many specimens was caused by cracks that initiated and propagated to failure outside the test section for the hourglass-shaped specimens (for example, fillet and radius regions of the transition from the test section to the shoulder, and in the shoulder and grip regions of the specimen). The fatigue life for these specimens was influenced by specimen preparation, specimen design, and test procedure, and it should not be used to characterize the behavior of unweathered or weathered specimens. Eliminating these test results from the total population presented in Figures 4 and 5 significantly decreases the number of data points that fall below the category A fatigue-design curve (Figures 6-8). Nevertheless, a few test results for both unweathered and weathered specimens still had fatigue lives that were lower than predicted by the design curve.

The data presented in Figures 6-8 represent the total population of test results for specimens whose failure was confined to the test (necked-down) section and for some specimens whose fracture origin was not identified and could have failed outside the test section. The data in Figures 7 and 8 also represent test results obtained from the three exposure sites that had significantly different atmospheric severity. One of the three sites, the Simonoseki site, was a marine environment and "was close to a tunnel where the exhaust from the vehicles [localized acid water] could have affected the results" (11). Unfortunately, the reported data were not identified by exposure site and, therefore, the effect of atmospheric severity on the fatigue life of the specimens, especially those exposed at the Simonoseki site, cannot be delineated.

Some of the unweathered-specimen test data fell below the AASHTO category A fatigue-design curve. Further analysis of the data presented by Kunihiro et al. (11) indicated that the yield strength for the SMA 50 grade steel varied between 47 and 64 ksi, and for the SMA 58 grade steel it varied between 67 and 74 ksi (the SMA 50 and 58 represent minimum tensile strengths in kg/mm^2). The minimum stress for the fatigue tests was about 3 ksi. Consequently, the maximum stress for specimens subjected to stress ranges that exceeded 44 ksi would be higher than the yield strength of some specimens. The fatigue life for specimens subjected to such test conditions, as

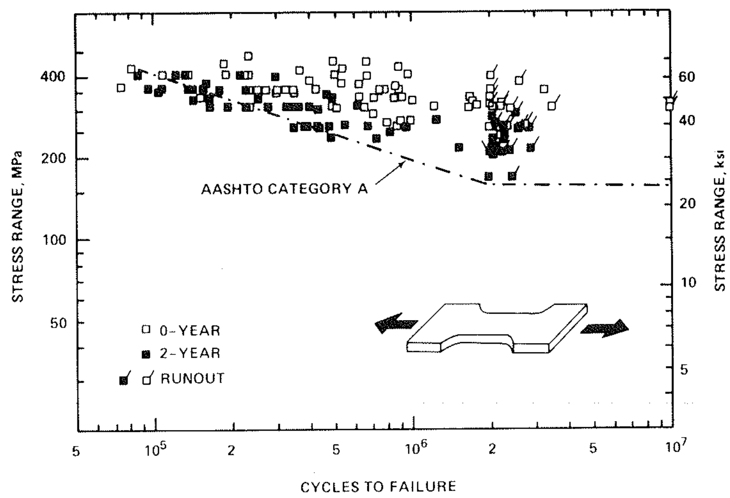

FIGURE 4 Fatigue strength of 2-year weathered plain plate specimens fabricated from Japanese atmospheric corrision resisting steels (SMA 50 and SMA 58).

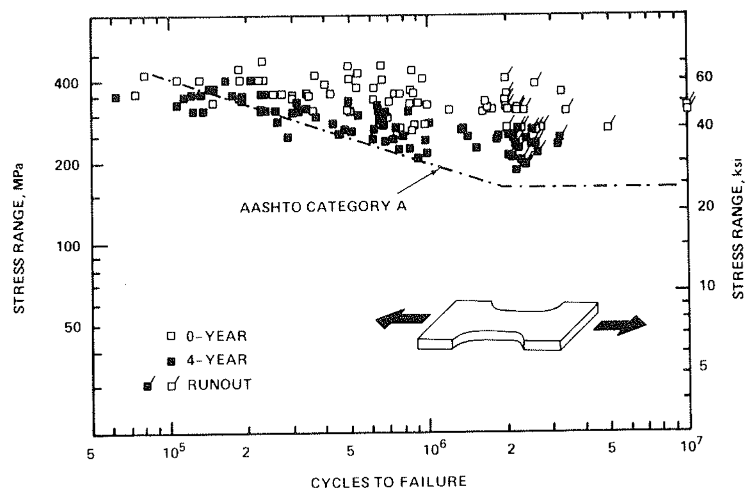

FIGURE 5 Fatigue strength of 4-year weathered plain plate specimens fabricated from Japanese atmospheric corrosion resisting steels (SMA 50 and SMA 58).

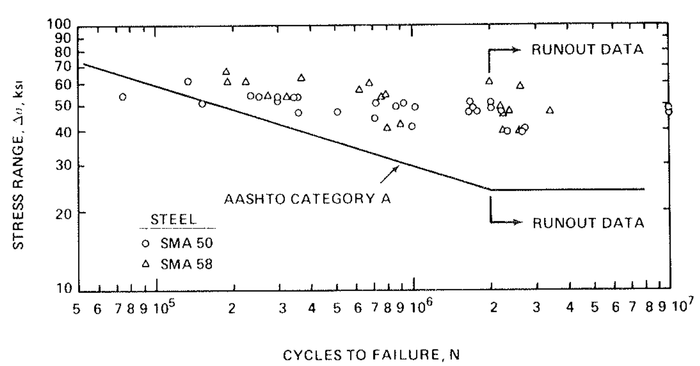

FIGURE 6 Fatigue data for plane specimens before atmospheric exposure.

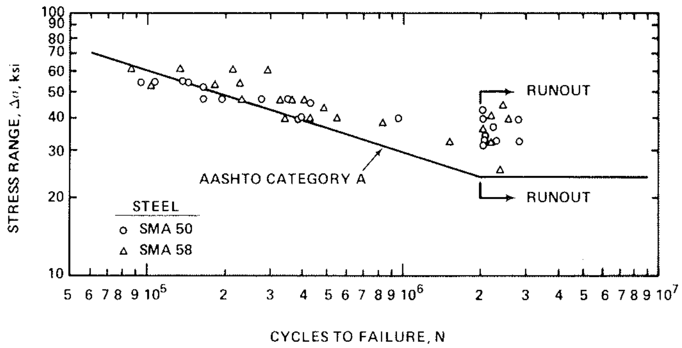

FIGURE 7 Fatigue data for plane specimens after 2-year exposure in three atmospheric environments.

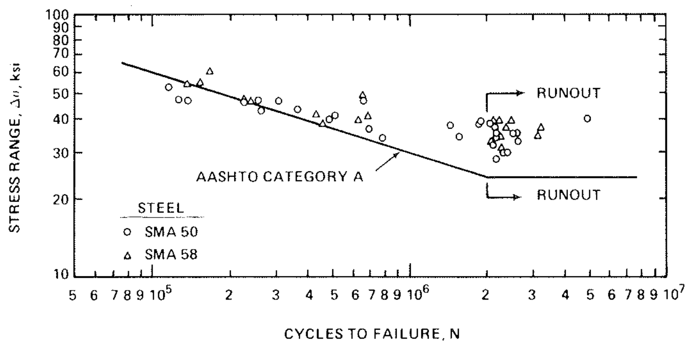

FIGURE 8 Fatigue data for plane specimens after 4-year exposure in three atmospheric environments.

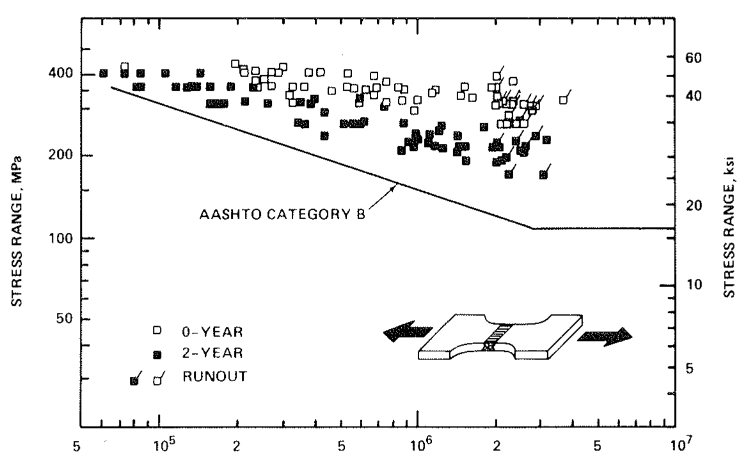

FIGURE 9 Fatigue strength of 2-year weathered butt-welded ground-flush specimens fabricated from Japanese atmospheric corrosion resisting steels (SMA 50 and SMA 58).

expected, is less than that predicted by the category A fatigue-design curve. In other words, the AASHTO design curve should not be used to characterize the behavior of these specimens. Unfortunately, although the yield strength for the steels that were obtained from different sources was reported, the source or yield strength for the individual fatigue specimens was not given.

Examination of the data presented in Figures 6-8 indicates that 10 of the 13 test results that exhibited fatigue lives shorter than those predicted for the AASHTO category A fatigue-design curve were of the lower-strength grade steel (yield strength ≥ 47 ksi). Nine of these 10 test results (two in Figure 6, four in Figure 7, and three in Figure 8) were at stress ranges (Δσ) equal to or higher than 47 ksi, with one (Figure 8) at a Δσ of about 55 ksi. The corresponding maximum stress for these specimens was equal to or higher than 50 ksi, with one subjected to a maximum stress of 58 ksi. The remaining specimen (1 of the 10 in Figure 8) was subjected to a stress range of 43.5 ksi and a maximum stress of

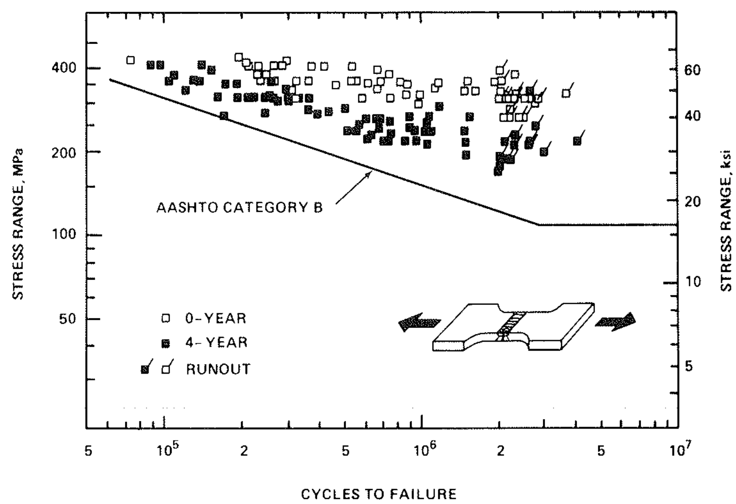

FIGURE 10 Fatigue strength of 4-year weathered butt-welded ground-flush specimens fabricated from Japanese atmospheric corrosion resisting steels (SMA 50 and SMA 58).

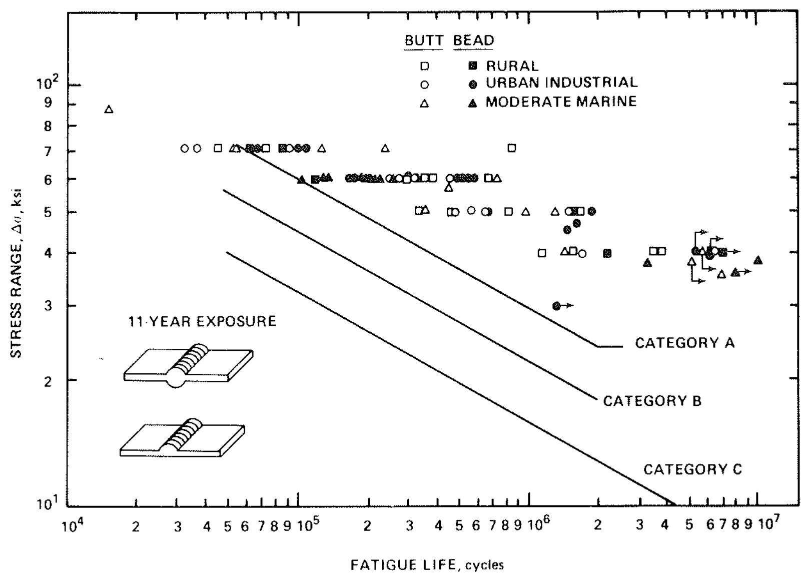

FIGURE 11 Fatigue data for butt-welded specimens and bead-on-plate specimens of ASTM A588 steel weathered for 11 years in three environments.

46.5 ksi, which is close to the minimum yield strength reported for the SMA 50 grade steel. The fatigue life for this specimen was 2.65 x 10^5 cycles, which is within 8 percent of the 2.85 x 10^5 cycles expected from the category A fatigue-design curve.

Based on the preceding observations and because the AASHTO fatigue-design curves represent the 95 percent confidence limit for 95 percent survival of all the details in a given category, the behavior for the data reported by Kunihiro et al. (11) appears to be consistent with the data base used in the development of the AASHTO fatigue-design curves.

Category B

Fatigue cracks in components that contain category B type details occur primarily at subsurface imperfections such as gas pockets. These imperfections are more severe than the surface or flame-cut edge imperfections allowable for category A type details. Because weathering is a surface phenomenon, it cannot alter the severity of subsurface imperfections. Consequently, the category B fatigue-design curve corresponds to a lower bound for weathered and unweathered category B details. These observations are supported by all available data (11,12) for weathered weathering steels (Figures 9 and 10), where all the test results for weathered and unweathered butt-welded ground-flush specimens exhibited longer fatigue lives than those predicted by the category B fatigue-design curve.

Categories C, D, and E

Fatigue cracks in specimens that correspond to categories C, D, and E initiate from similar weld imperfections that are equal to or smaller than 0.016 in. and that are located at weld toes and weld terminations. The decrease in fatigue performance from

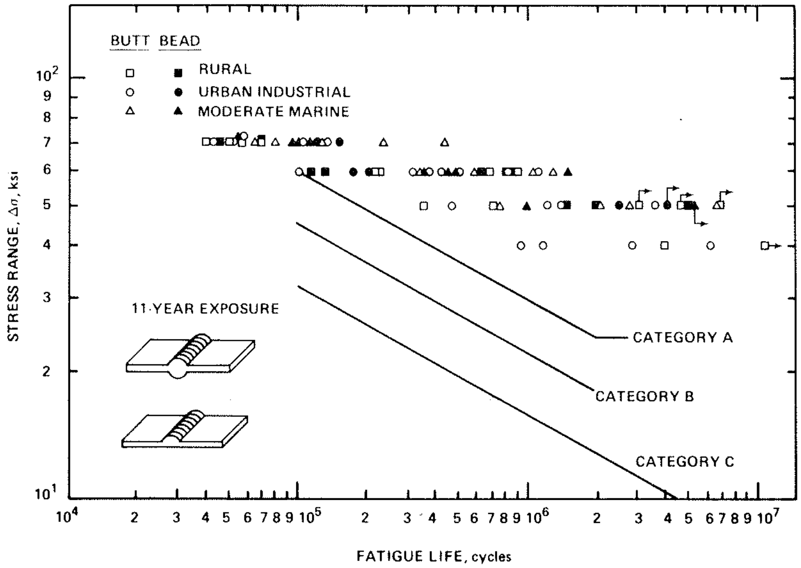

FIGURE 12 Fatigue data for butt-welded specimens and bead-on-plate specimens of ASTM A242 steel weathered for 11 years in three environments.

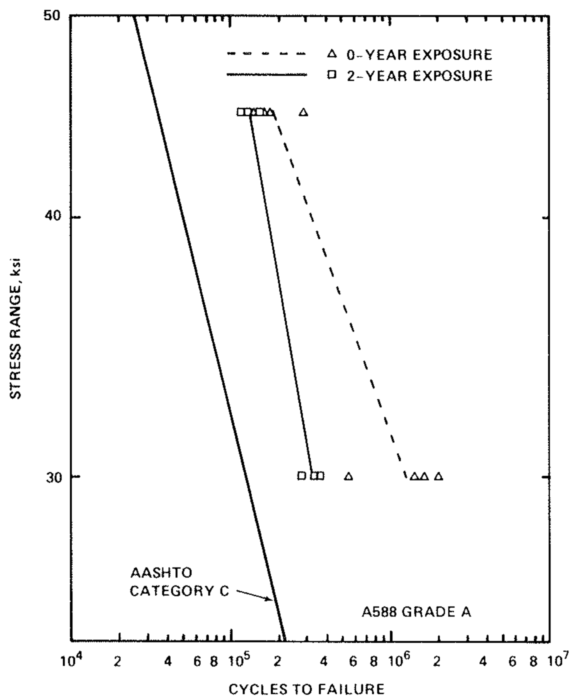

FIGURE 13 Fatigue data for blast-cleaned welded specimens (from Blake, unpublished data).

category C to D to E is related primarily to an increase in the geometrical stress concentration in the regions of the weld imperfections. Because metal removal due to weathering is negligible when weathering steel is used properly, and because weathering does not preferentially attack the weld metal or heat-affected zone over the base metal (13), it cannot increase the geometrical stress concentration inherent in the geometries for categories C, D, and E. Furthermore, because the size (0.016 in.) of the weld imperfections for these categories is small, the removal of a surface layer of metal by weathering may eliminate or at least decrease the size of the surface imperfections, thus resulting in a possible improvement in the fatigue lives for category C, D, and E details that initially contained severe imperfections. Improved fatigue life for fabricated components as a result of weathering has been documented by Yamada (14).

Figures 11-15 present fatigue data for unweathered and weathered weathering-steel category C details (12, 15, and unpublished data from Blake). These figures present data from specimens that have been weathered for up to 11 years. The combined data indicate that weathering had a negligible effect on the fatigue behavior of category C type details. More importantly, all the specimens exhibited better fatigue lives than those that correspond to the AASHTO category C fatigue-design curve.

Figure 16 (12) presents fatigue data for unweathered and weathered weathering-steel category D details. The data indicate that the fatigue behavior of weathered and unweathered specimens are identical, and that the AASHTO category D fatigue-design curve predicts their behavior conservatively.

No data are available for weathered category E type specimens; however, based on the preceding observation, weathering should not have any adverse effect on their fatigue behavior.

Based on all available data for the fatigue behavior of weathered weathering-steel components and on the discussion in the preceding sections, it can be concluded that the current AASHTO fatigue-design curves are equally applicable to predict the fatigue behavior of weathered as well as unweathered bridge-steel components.

SUMMARY

The data and discussion presented in this paper indicate that surface roughness of steels caused by weathering corresponds to localized stress (strain) raisers on the surface that may decrease the fatigue life of weathered components. Consequently, the effect of weathering on fatigue life is more pronounced for category A details of the AASHTO fatigue-design provisions. Nevertheless, because the most severe surface imperfections that correspond to the various AASHTO fatigue-design categories are more severe than those generated by weathering, the

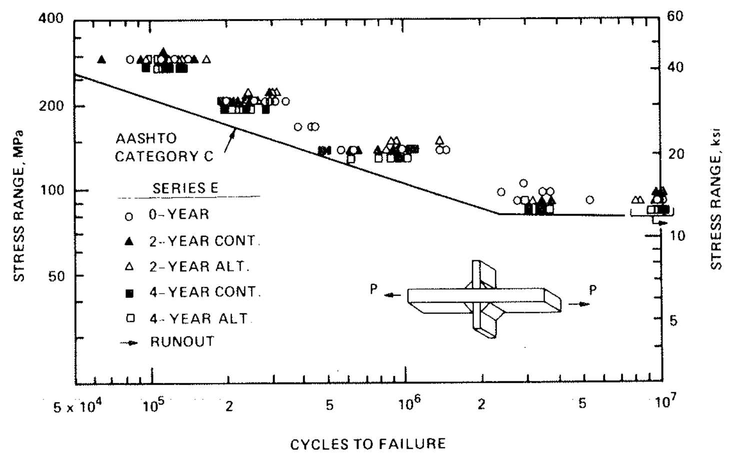

FIGURE 14 Comparison of data for specimens with transverse stiffeners and AASHTO allowable category C line (12).

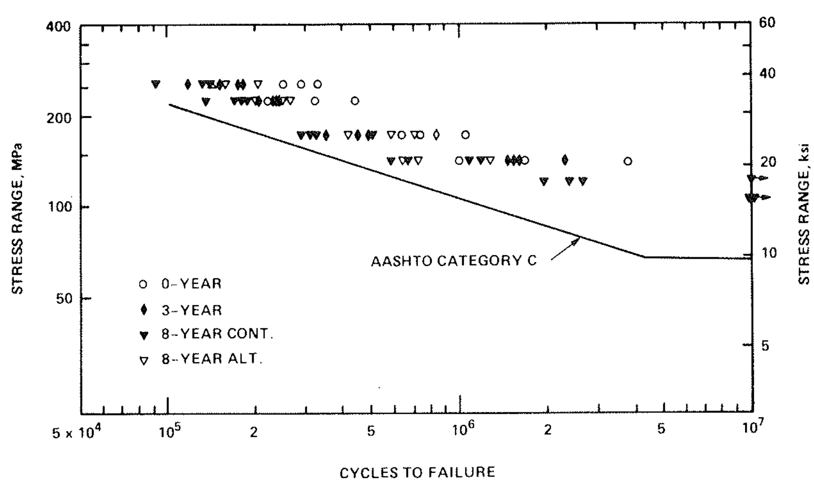

FIGURE 15 Comparison of category C type specimens and AASHTO allowable category C curve (15).

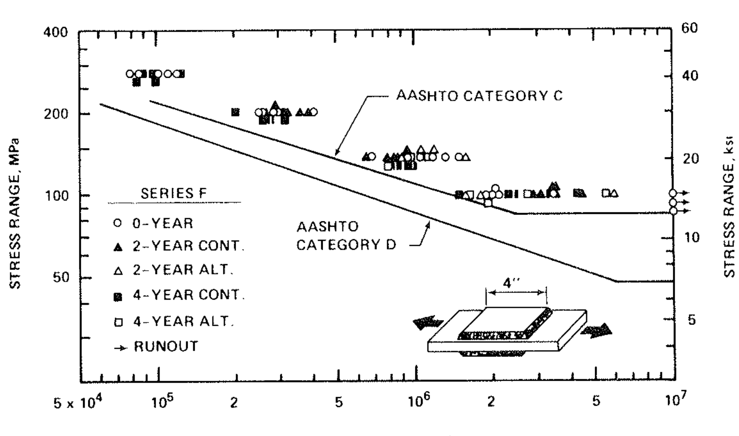

FIGURE 16 Comparison of all data for specimens with attachments (12).

current AASHTO fatigue-design curves should be equally applicable to predict the fatigue behavior of weathered as well as unweathered structural-steel components.

REFERENCES

1. Standard Specifications for Highway Bridges. AASHTO, Washington, D.C., 1977.
2. Surface Texture. Report ANSI B 46.1-1978. American National Standards Institute, New York, 1978.
3. J.W. Fisher, K.H. Frank, M.A. Hirt, and B.M. McNamee. Effect of Weldments of the Fatigue Strength of Steel Beams. NCHRP Report 102. TRB, National Research Council, Washington, D.C., 1970, 114 pp.
4. J.W. Fisher, P.A. Albrecht, B.T. Yen, D.J. Klingerman, and B.M. McNamee. Fatigue Strength of Steel Beams with Welded Stiffeners and Attachments. NCHRP Report 147. TRB, National Research Council, Washington, D.C., 1974, 85 pp.
5. F.C. Lea and J.G. Whitman. The Failure of Girders Under Repeated Stresses. Welding Research Supplement, Vol. 18, No. 1, Jan. 1939.
6. J.D. Nee. Fatigue Strength of USS "T-1" Constructional Alloy Steel Beams With and Without Stiffeners. Applied Research Laboratory, United States Steel Corporation, Monroeville, Pa., Feb. 1966.
7. D.R. Sherman and J.E. Stallmeyer. Fatigue of "T-1" Beams. Status Report of Fatigue Committee. Welding Research Council, University of Illinois, Urbana, May 1963.
8. W.M. Wilson. Flexural Fatigue Strength of Steel Beams. Engineering Experiment Station Bull. 377. University of Illinois, Urbana, Vol. 45, No. 33, Jan. 1948.
9. E.G. Signes et al. Factors Affecting the Fatigue Strength of Welded High Strength Steels. British Welding Journal, Vol. 14, No. 3, 1967.
10. F. Watkinson et al. The Fatigue Strength of Welded Joints in High Strength Steels and Methods for Its Improvement. In Proc., Conference on Fatigue of Welded Structures, The Welding Institute, Brighton, England, July 1970.
11. T. Kunihiro, K. Inoue, and T. Fukusa. Atmospheric Exposure Study of Weathering Steel. Research Laboratory Report 729. Ministry of Construction, Tokyo, Japan, 1972.
12. P. Albrecht. Fatigue Behavior of 4-Year Weathered A588 Steel Specimens with Stiffeners and Attachments. Report FHWA/MD-81/02. Department of Civil Engineering, University of Maryland, College Park, July 1978.
13. P.R. Simmon. Arc Welding of Weathering Steels. Welding Journal, Dec. 1968.
14. K. Yamada. Japanese Experience on Weathering Steel Bridges. Department of Civil Engineering, Nagoya University, Furo-Cho, Chikusa-Ku, Nagoya, Japan, 1983.
15. P. Albrecht and J.G. Cheng. Fatigue of 8-Year Weathered Automatically Welded A588 Steel Stiffeners. Department of Civil Engineering, University of Maryland, College Park, June 1982.

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Fatigue Strength of Weathered and Deteriorated Riveted Members

JOHANNES M.M. OUT, JOHN W. FISHER, and BEN T. YEN

ABSTRACT

A study has been performed on the fatigue resistance of corroded and deteriorated riveted members. The need for this study arose from the concern with the large number of riveted structures functioning today that have various degrees of corrosion and potential fatigue damage. The validity of AASHTO and American Railway Engineering Association category D that is generally used for riveted connections is uncertain, particularly near the fatigue limit. A series of fatigue tests was carried out on 80-year-old steel bridge stringers with a riveted built-up cross section. The stringers were significantly corroded along the compression flange and locally at the tension flange. The stress ranges that were applied were selected between the fatigue limits of design categories C and D. The corroded region of the tension flange proved to be the most severe condition, varying between categories C and E. The category D fatigue limit appears to be applicable to the rivet detail studied. The reduction of the compression flanges had no effect on the performance of the member. A strong frictional bond between section components was found to have a beneficial effect on fatigue life. A