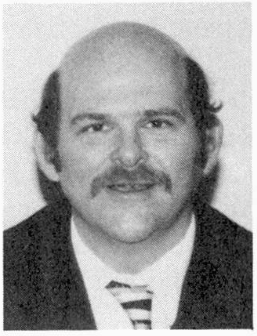
Fatigue Cracking of Orthotropic Steel Decks

Robert J. DEXTER
Senior Research Eng.
Lehigh University
Bethlehem, PA, USA

Robert J. Dexter, born 1956, got his Ph.D. in Civil Engineering at the University of Texas at Austin. He has over 16 years experience in research on welding, fatigue, and fracture.



John. W. FISHER
Professor
Lehigh University
Bethlehem, PA, USA

John W. Fisher, born 1931, got his Ph.D. in Civil Engineering at Lehigh University and is currently director of the ATLSS Center. He is a member of the National Academy of Engineering.

Summary

A review is presented of the service fatigue-cracking problems with orthotropic steel decks in the last fifteen years; as well as available sources of fatigue test data and U.S. design recommendations. The causes of these cracking problems and the repair and retrofit procedures that were used to correct the problems are discussed. Many distortion-induced cracks can be retrofit at relatively low cost by hole drilling. Recommendations are made for fatigue-resistant weld and cutout details for future designs. A recently conducted full-scale test of a cantilevered orthotropic deck is discussed, along with recommendations for future research.

1. Introduction

The initial cost of an orthotropic steel deck is at least twice the initial cost of a 228 mm thick concrete deck. However, orthotropic steel decks are lighter and, provided they are designed to resist fatigue cracking, more durable than concrete decks. Orthotropic steel decks are also attractive for deck replacement because they can lower dead load on deteriorated superstructures and can be replaced in modules during temporary lane closures.

Structural elements in the bridge deck have very little dead load. If there are a large number of live load cycles, the fatigue limit state will generally govern the design. Bridge deck elements are loaded not just by every truck, but by every axle. Within a few years, the number of cycles of bridge deck elements can become very large, in most cases larger than the number of cycles associated with the constant-amplitude fatigue limit (CAFL) for the welded details.

Fatigue cracks have been observed in various details of orthotropic steel decks in the last fifteen years. This problem has been most severe in England, Germany, Austria, France, Japan, and Australia, particularly because thin deck plates (8 to 12 mm) were used. For the most part, the fatigue-cracking problems reflect: 1) the limited fatigue-test data on orthotropic deck systems; and, 2) the over-optimistic definition of the fatigue strength that was attributed to these details in the 1960's and 1970's. The CAFL for most of these details was not known as relatively few tests were carried out beyond two million cycles. Hence, most of the details are subjected to service stress ranges that exceed the CAFL. Consequently, fatigue cracks develop after some time, perhaps not until after ten or more years of service.

Experimental studies of the fatigue strength of orthotropic deck details were summarized in 1989 by the Office for Research and Experiments (ORE) of the International Union of Railways in Utrecht, Netherlands [1]. This report described fatigue critical orthotropic deck details and their the fatigue strength. A second summary report was prepared by the Commission of European Communities (ECSC) [2]. This report summarizes testing and analyses from Germany, Belgium, France, Italy, Holland, and England.

2. Fatigue Design Provisions for Long-Life Variable-Amplitude Loading

The American Association of State Highway Transportation Officials (AASHTO) LRFD Bridge Design Specifications [3] contain specific guidance on the fatigue design of orthotropic steel decks. The design fatigue resistance categories of orthotropic deck details in the AASHTO LRFD recommendations are essentially the same as the recommendations in the ORE report, although there are slight differences in the fatigue strength of some details.

Typically, this fatigue design stress range is obtalned from a static analysis where the wheel loads are applied in patches, and the axle load is equal to an impact factor times the design axle load. In the AASHTO LRFD specifications, an impact factor of 1.75 is recommended for deck elements. There is a great deal of uncertainty in the impact factor and the size of the wheel-load patches [4-8]. Also, due to calibration of the AASHTO LRFD specifications against previous specifications and experience, the fatigue design axle load is in fact much less than the 1:10000 exceedance level [4]. The discrepancy has been defended because it is felt that other aspects of the design process are overconservative, such as the assumptions in the structural analysis models.

The fatigue design philosophy in the U.S. for long-life (i.e. for numbers of cycles greater than the number of cycles associated with the CAFL) is to require that essentially all the stress ranges are less than the CAFL. Variable-amplitude fatigue tests on full-scale girders with welded details show that if less than 1:10,000 cycles exceed the CAFL, then essentially infinite life is obtained [9]. This approach is the basis of provisions in the AASHTO LRFD specifications, and has also been applied in developing fatigue design specifications for expansion joints [4-8], which are also loaded by every axle, and wind-loaded sign, signal, and luminaire support structures [10]. One advantage of this approach for structures with complex stress histories is that it is not necessary to accurately predict the entire future stress range histogram. The fatigue design procedure requires only the stress range with an exceedence level of 1:10,000, which is called the fatigue-limit-state stress range.

The U.S. approach for long-life variable-amplitude fatigue requires accurate definition of the CAFL. The upper part of the S-N curve is only needed for situations where the number of cycles is less than the number of cycles associated with the CAFL. These situations are referred to as being in the "finite-life" regime. For deck elements and other elements subjected to long-life loading, the emphasis in fatigue testing of details must be on defining the CAFL, which requires more expensive long-term testing at stress ranges close to service stress ranges.

The AASHTO LRFD code and the Eurocode both use an effective stress range for variable-amplitude loading (defined by Miner's rule) for use with the constant-amplitude S-N curves. However, there are large differences between the Eurucode and the AASHTO LRFD code regarding the CAFL. In the development of the U.S. codes, the CAFL has been defined as the largest stress range for which all fatigue tests are terminated with no cracking. The number of cycles associated with the CAFL is whatever number of cycles corresponds to that stress range on the S-N curve for that category or class if detail. The CAFL occurs at an increasing number of cycles for lower fatigue categories or classes. Sometimes, different details, which share a common S-N curve (or category) in the finite-life regime, have different CAFL.

The Eurocode S-N curves have a CAFL at five-million cycles regardless of the class. For variable-amplitude loading, the Eurocode S-N curves have a change in slope below the CAFL, with a cutoff at 100 million cycles.

Since both approaches are based on experimental data, it is not surprising that both result in approximately the same design for given fatigue loads [8]. Since these two approaches use different stress ranges, it is necessary to estimate a relationship between these stress ranges. Typical truck axle load spectra are such that the fatigue-limit-state stress range is typically about two times the effective stress range.

For example, using the Eurocode and considering a class 90 detail (AASHTO category C), the effective stress range should be just below the fatigue strength at 100 million cycles, which is about 40 MPa. Using the AASHTO LRFD approach, the fatigue limit-state stress range should be just below the CAFL, which is about 70 MPa for the AASHTO Category C.

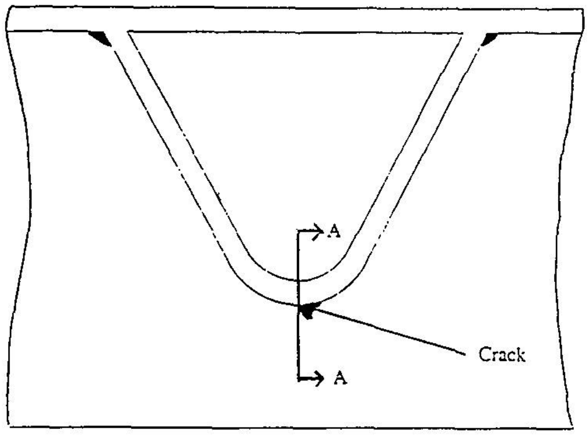
3. Types of Fatigue Cracking Observed on Orthotropic Steel Decks

Generally, fatigue-cracking problems can be classified as either: 1) load-induced cracking; or, 2) distortion-induced cracking; depending on the boundary conditions of the loading which causes the stress range driving the cracking. Orthotropic steel deck cracking problems have included both types.

3.1 Load-induced cracking

The AASHTO LRFD bridge design specifications [3] cover most cases of load-induced cracking of orthotropic steel decks. Load-induced cracking is a result of the fluctuation of the nominal primary stresses, i.e. the stress range induced by the applied wheel loads using standard first-order design calculations. Load-induced cracking occurs primarily at poor details when these details are subjected to significant stress ranges exceeding the CAFL.

For example, in some early orthotropic decks, longitudinal ribs were butted up to the floor beams and fillet welded all around, as shown in Figure 1. When tension developed in the ribs, the unfused parts of the rib/diaphragm interface acted like a notch and easily cracked through the weld root. The ORE report rated the fatigue strength of this detail as the equivalent of the AASHTO Category E' with respect to the nominal longitudinal bending stress range in the rib. The cracking of this detail led to the present practice of passing the rib continuously through the diaphragm. The AASHTO LRFD code only shows the continuous-rib detail and rates the fatigue strength of this preferable detail as Category D with respect to the nominal longitudinal bending stress range in the rib.



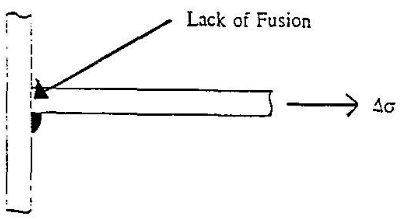


Figure 1: Fillet-welded connection of rib which is not continuous through the diaphragm

Other examples of load-induced cracking at poor details have been related to welded field splices of the longitudinal ribs. Unusually large defects in the rib splice welds have resulted in cracking. Backing bars left in place inhibit good ultrasonic testing. The AASHTO LRFD code rates the welded rib splice with a backing bar as a Category E detail. The orthotropic deck of the Rio-Niteroi Bridge in Brazil has exhibited hundreds of fatigue cracks. About 94 percent of the cracking is associated with the field-welded splices between the 15 m sections. Stress ranges were measured in these ribs in excess of the CAFL for Category E details (31 MPa).

3.2 Distortion-induced cracking

Distortion-induced cracking results from second-order stresses, typically due to out-of-plane deformations and incompatible deformations at intersecting structural elements. The stresses that cause this type of cracking are very difficult to quantify. Even detailed finite-element analyses typically cannot accurately calculate these distortion-induced stresses. Since an accurate calculation-based design method is presently not available, control of distortion-induced cracking is accomplished through the art of good detailing in addition to the standard workmanship requirements and quality control. In the AASHTO LRFD code, control of distortion-induced cracking is affected through: 1) design guidance requiring rigid load paths for possible secondary forces (such as in diaphragms); and, 2) prescriptive requirements such as minimum plate thickness.

For example, longitudinal ribs were originally welded to the deck plate using one-sided fillet welds. Figure 2 shows the transverse moments due to distortion that occur at this connection, causing prying of the unfused notch and cracking through the weld root. This problem occurred in a suspension bridge in the United Kingdom and was extensively studied by Gurney et al at The Welding Institute. Virtually all of these welds had to be gouged and rewelded. These longitudinal welds are now required to have at least 80 percent penetration. Also, the LRFD code requires the deck plate to be 16 mm or greater, and has requirements for the maximum rib thickness. The deck plate requirements assure that the distortions are minimized, and the rib thickness limits insure that the rib is flexible enough to accommodate the rotations.

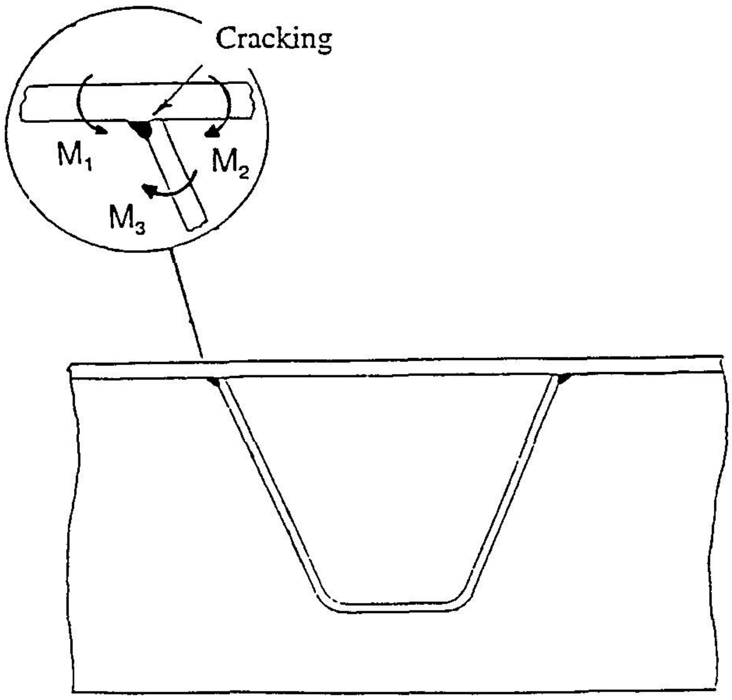


Figure 2: Distortion-induced cracking at longitudinal fillet-weld joining rib to deck plate

Distortion-induced cracking can also occur for a number of reasons in the connection of the transverse diaphragms to the continuous ribs. One such problem is due to shear deformations in the diaphragms and is particularly severe in cantilever sections due to high shear forces on tapered floor beams. Also, slow traffic lanes, with the heaviest trucks, are typically located on the ends of these cantilevers. There are some explicit design calculations for this shear problem in the AASHTO LRFD code.

Figure 3 shows some of the cracks which might occur at the diaphragm/rib connection including cracking in the cutouts and longitudinal cracking of the ribs at the intersection with the cutout. In 1990, similar longitudinal cracks were noticed in the ribs of the Westgate Bridge in Australia [11]. The ribs did not have internal diaphragms or bulkhead plates to carry the forces imparted by the floorbeam diaphragm. These cracks originated from "oil-can" deformation of the ribs by the floor-beam diaphragms. Internal bulkhead plates are now considered good practice. These bulkhead plates should be at least 12 mm thick with 8 mm fillet welds, and should be almost full depth of the rib, i.e. the bulkhead should come up as close as possible to the top of the rib and extend down as close as possible to the bottom of the rib, considering required clearance and tolerances.

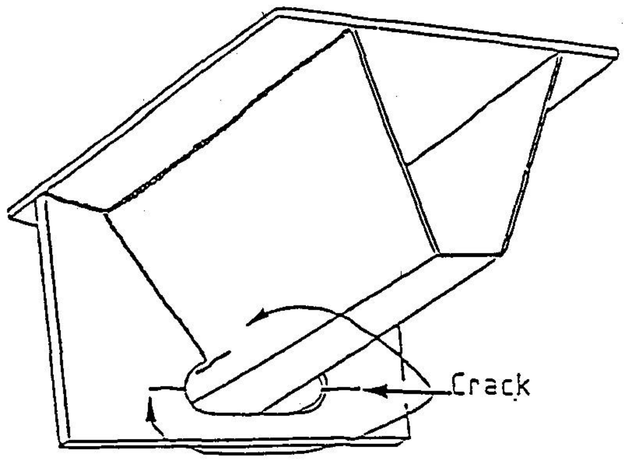


Figure 3: Diaphragm-to-rib connection showing typical crack locations at the cutout

Perhaps the most critical aspect of this diaphragm/rib connection is the termination of the fillet welds. The welds should not wrap around the diaphragm, rather, the ends of the welds should be ground. Figure 4 shows the traditional fillet weld termination detail as well as a new combination weld detail for the weld termination which was developed by Steinman Consulting Engineers for the replacement deck for the Williamsburg Bridge in New York City. The combination weld detail includes a partial length complete-penetration groove weld which is ground to a smooth transition at the intersection of the cutout. New York City Department of Transportation had a full-scale fatigue test of the proposed deck design, with both the traditional detail and the combination weld detail, conducted at the ATLSS Center at Lehigh University [12]. The unique loading scheme to simulate rolling traffic loads was developed by New York City DOT [13]. Figure 5 shows a view of the top of the deck which is on floorbeams which are cantilevered from strong wall at the ATLSS laboratory. Figure 6 shows a close-up view of a typical diaphragm with the strain gages to measure the complex distortion-induced stress ranges near the rib connection. The tests showed the improved fatigue resistance of the Option A detail.

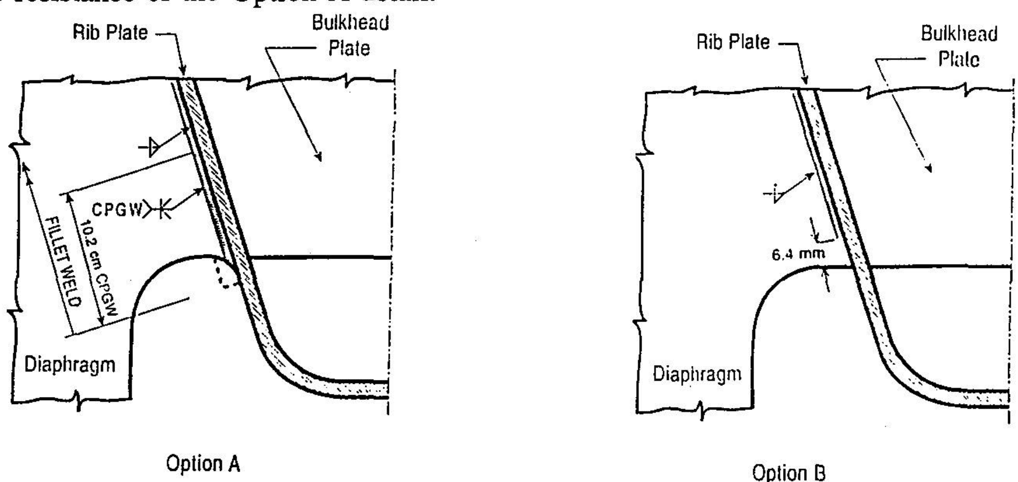


Figure 4: Alternative details for the termination of the weld in the diaphragm-to-rib connection

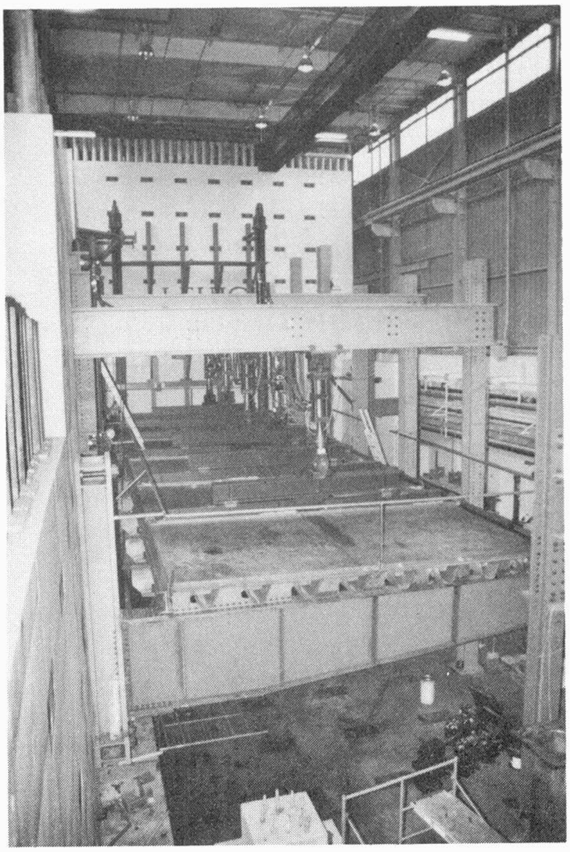


Figure 5 Full-scale fatigue test of a cantilevered section of an orthotropic deck.

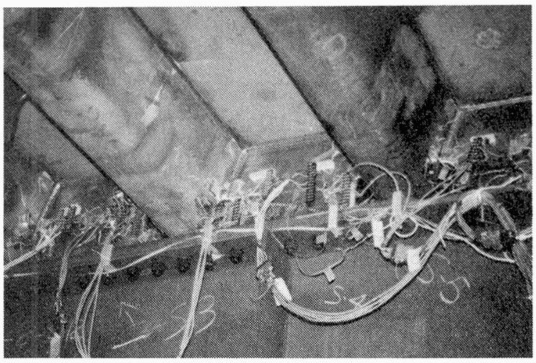


Figure 6: Typical diaphragm of the full-scale test showing strain gage locations near the rib connection.

Other recent test data, some of which are included in the ECSC report, show that fully welded details with no cutout have greater fatigue resistance than details with the cutout. Unfortunately, fit-up problems are more difficult without the cutout, therefore the decision may be made to use the cutouts despite the lower fatigue resistance.

The LRFD code has some guidance on cope holes and cutouts to reduce problems with cracking. Copes or snipes in the corner, where the diaphragm is welded to both the rib and the deck plate, are not allowed because of known fatigue problems. The distance between the bottom of the ribs and the connection to the diaphragm should be maximized.

In the United States, orthotropic decks are mostly used in redecking older bridges. A special type of distortion-induced cracking associated with redecking is caused by incompatibility between the curvatures of the rigid deck and the rather flexible floor systems. This problem usually manifests as longitudinal rotation and prying of the connection between the deck and the floorbeams (or stringers in some cases). This type of cracking has been observed on the Throg's Neck Bridge, for example. The problem was recently studied using finite-element analysis at Weidlinger and Associates.

4. Repair and Retrofit

There are a few cases where a distortion-induced crack is reasonably small, e.g. less than 75 mm, and is propagating only under the influence of secondary transverse stress ranges. These stress ranges are due to local distortion of the orthotropic deck under wheel loads. The stresses that result from the distortion decrease locally as the cracks make the section more compliant. Therefore, many of these cracks can probably be left in place after holes are drilled or cored at the tips of the crack. Coring at the tip of a crack essentially blunts the tip of the crack. Cracking will not reinitiate if the size of the hole satisfies the relationship:

$$\frac{\Delta K}{\sqrt{\rho}} \leq 10.5\sqrt{\sigma_y}$$

where ρ is the radius of the hole, mm, and σ_y is the yield strength of the plate, MPa. ΔK is the range in the stress intensity factor (MPa-$m^{1/2}$) determined from fracture mechanics analysis from the applied stress range and the crack size. The hole size required by this equation is reasonable for small stress ranges. For example, for 350 MPa yield strength and a typical crack geometry, the equation requires a hole diameter about 20 percent of the crack length for a stress range of 35 MPa. However, the dependence on stress range is strong. For the same conditions, the hole diameter must be 80 percent of the crack length for a stress range of 70 MPa. For higher stress ranges, the entire crack must be removed. Care must be taken that the hole is not a critical stress concentration in the member.

In practice, especially in these orthotropic deck cracks, the definition of the stress range and ΔK may not be possible. In this case, conservative experience-based rules of thumb must be relied upon. Most small cracks less than 50 mm long can be successfully retrofit using a 19 mm diameter hole.

For cracks which are not entirely longitudinal or for other reasons cannot be left in place, the preferred repair procedure is to gouge out the crack and fill the groove with weld metal. When a crack is to be gouged out by the air-arc process, it is a good idea to drill holes at the tips of the crack before gouging. For fillet welds which have cracked through the throat, the size of the repair weld should be increased relative to the original weld. Inadvertent gouges created by maintenance equipment may also be filled with weld metal. Small surface cracks can often be removed with a pencil grinder or disc grinder.

There are many cases where additional retrofit is required to upgrade the details, which should be applied to cracked as well as uncracked details. Cracks have occurred at bolted flanges in end plates due to prying because of the inadequate thickness of the end plates. After the cracks are drilled and weld repaired, thick doubler plates or angles should be added to these end plate connections to stiffen them.

Cracks have also occurred at lap welds for splice plates which are lapped over the sides and bottom of the trapezoidal stiffeners. These welds can be air-hammer peened. The peening may be performed only at the outer ends of the side plates and the bottom cover plate. The peening should wrap around the corner and continue for about 25 mm away from these ends. Light surface grinding following the peening improves the fatigue resistance further. Peening can repair shallow surface cracks less than a few millimeters deep. Visible cracks greater than a few millimeters deep must be repair welded.

Another example of a retrofit is the short segments of flat-bar stiffener that have been welded in some cases to the bottom of the trapezoidal stiffeners (fins). A core should be taken about 25 mm in diameter at each end of these stiffeners. The cores should include the end of the flat bar and both fillet weld ends. The core diameter may be increased within reason to remove entirely existing cracks at these stiffener ends. Cracks which extend beyond the core diameter should be repair welded prior to coring.

Another consideration is the design and maintenance of the asphalt wearing surface. Potholes and severe deterioration of the pavement can increase the dynamic wheel loads and significantly decrease the fatigue life of the orthotropic steel deck. New asphalt high-performance wearing surfaces have been developed in Europe. These types of surfaces have a tack course applied to the steel surface and a thin wearing surface on top. The experience base with this type of surface is limited, but it was used on the Normandie Bridge in France which was recently completed.

5. Conclusions

Orthotropic steel decks have had a number of fatigue cracking problems in service which can be classified as either load-induced cracking or distortion-induced cracking. Design to resist load-induced cracking problems is relatively straightforward. Most of these problems have occurred because of inadequate knowledge of the fatigue strength of the details. The correction of service load-induced cracking problems typically involves upgrading the detail, which can be very expensive.

Distortion-induced cracking problems are typically attributable to unanticipated secondary stresses. Accurately predicting the applied stress range in design for the case of distortion-induced cracking is usually not possible. Therefore, distortion-induced cracking problems are controlled through minimum plate thickness and other detailing requirements. Distortion-induced cracks can usually be retrofit at relatively low cost, e.g. by hole drilling.

6. Acknowledgements

The paper is based on case studies investigated by the authors, cases described in the literature, and cases discussed with other engineers. The authors appreciate the support of New York City Department of Transportation, New York State Department of Transportation, and Federal Highway Administration for the testing of the prototype deck for the Williamsburg Bridge. This testing of the deck at ATLSS was conducted by Research Engineer Mark R. Kaczinski, presently at The D.S. Brown Company in North Baltimore Ohio, and Research Assistant Peter Lugger, presently at Steinman Consulting Engineers in New York City. The deck sections were fabricated by Leonard Kunkin Associates of Lexington, PA.

7. References

1. Office for Research and Experiments (ORE) of the International Union of Railways, "Fatigue Calculations for Orthotropic Decks of Steel Railway Bridges", Report No. 5, ORE, Utrecht, April 1989.

2. A. Bruls, ed., "Measurements and Interpretation of Dynamic Loads in Bridges. Phase 3: Fatigue Behavior of Orthotropic Steel Decks. Common Synthesis Report", Commission of European Communities (ECSC), Directorate - General XII Science, Research and Development, March 1990.

3. American Association of State Highway and Transportation Officials Standard Specifications for Highway Bridges - LRFD, 1st Edition, 1994.

4. R.J. Connor, "Fatigue Criteria for Modular Expansion Joints", M.S. Thesis, Dept. of Civil and Environmental Engineering, Lehigh University, 1996.

5. M.R. Kaczinski, R.J. Dexter, and R.J. Connor, "Fatigue Design and Testing of Modular Bridge Expansion Joints", Proceedings of the Fourth World Congress on Joint Sealing and Bearing Systems for Concrete Structures, Sacramento, September 1996.

6. J.W. Fisher, R.J. Dexter, and M.R. Kaczinski, "Field and Laboratory Experiences with Expansion Joints", Structural Engineering in Consideration of Economy, Environment, and Energy, 15th Congress of IABSE, Copenhagen, Denmark, 16-20 June 1996.

7. R.J. Dexter, M.R. Kaczinski, and J.W. Fisher, "Fatigue Testing of Modular Expansion Joints for Bridges", IABSE Symposium San Francisco 1995, Extending the Lifespan of Structures, Vol. 73/2, pp.1091-1096, 1995.

8 J.A. Van Lund, M.R. Kaczinski, and R.J. Dexter,. "Modular Bridge Expansion Joints for the Lacey V. Murrow Floating Bridge", 1997 Annual Transportation Research Board Meeting, Washington, D.C., January 1997.

9. Fisher, J.W., et al, Resistance of Welded Details Under Variable Amplitude Long-Life Fatigue Loading, National Cooperative Highway Research Program Report 354, Transportation Research Board, Washington, D.C., 1993.

10. Kaczinski, M.R., Dexter, R.J., and VanDien, J.P., Fatigue-resistant Design of Cantilevered Signal, Sign, and Light Supports, Final Report on Project 10-38, National Cooperative Highway Research Program, Transportation Research Board, Washington, D.C., 1996.

11. P. Grundy, Measurement and Interpretation of Strains on Westgate Bridge, Monash University Report, Australia, December 1990.

12. Kaczinski et al., "Full-Scale Fatigue Test of the Williamsburg Bridge Orthotropic Deck", Building an International Community of Structural Engineers, Vol. 1, Proceedings of Structures Congress XIV, Chicago, p.329, April 1996

13. R.B. Gajer, J. Patel and D. Khazem, "Orthotropic Steel Deck for the Williamsburg Bridge Reconstruction", Building an International Community of Structural Engineers, Vol. 1, Proceedings of Structures Congress XIV, Chicago, pp. 491-498, April 1996