Distortion-Induced Cracking During Transit

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1. INTRODUCTION

During the last twenty years, there has been a growing awareness of distortion-induced cracking in bridges. Comparable interest in this behavior has not been generated within the building community. Distortion-induced fatigue cracking is indeed a more common problem in bridges, arising from cyclic distortions not only during handling and shipping but also during service. Bridge service conditions are much more likely to induce cyclic distortions than building service conditions. However, handling and shipping conditions are similar for both building and bridge components. It would therefore be beneficial if the topic of distortion-induced cyclic stress as a possible cause of fatigue cracking during shipment were more widely understood in the steel industry. This paper reviews the characteristics of distortion-induced cracking as seen in bridges, presents a detailed example of cracking in building trusses due to cyclic distortions during transit, summarizes the conditions that lead to this type of cracking during shipment, discusses the effect of cracking on structural performance, and provides recommendations for prevention of this type of damage.

2. DISTORTION-INDUCED CRACKING

Different kinds of steel structures and steel details are susceptible to cracking from different causes. There are two major classes of bridge and building fatigue damage[1]:

1. load-induced (also known as primary stress induced), and
2. distortion-induced (also known as secondary stress induced).

The cyclic stresses driving load-induced fatigue are due to the primary load bearing behavior of the structure under variable loads such as traffic for bridges and cranes for buildings. Stress-induced fatigue is the type of cracking behavior of which structural engineers are usually most aware. Bridge and building design specifications[2, 3, 4] address load-induced fatigue by classifying detail types according to expected cracking behavior. These classifications are used to specify appropriate design provisions intended to prevent fatigue crack failures. Distortion induced fatigue is driven by relatively small out-of-plane displacements caused by the mutual presence of an abrupt change in stiffness and a periodic force opposing it.[5] This type of cracking is most prevalent if, in addition to the required conditions of a discontinuity in stiffness subjected to a periodic distortion, there is also a weld toe located in the high cyclic stress region.[1] Current design code provisions only peripherally address distortion-induced cracking. The current AASHTO design specifications, for example, indirectly address distortion-induced cracking by requiring a rigid attachment of connection plates to both top and bottom flanges.

To understand distortion-induced cracking, it is useful to determine what conditions commonly lead to this type of cracking and then look at some specific examples. A survey[6,7] of 142 bridges that had developed cracking gathered information on 149 instances of primary cracking causes (several sites developed more than one type of cracking in different structural details). These 149 cases can be grouped into 28 categories of cracking.[1] These categories can be organized into a hierarchical classification of the most prevalent causes of cracking in steel bridges.[8] Figure 1 shows such a classification of the primary causes for cracking in steel bridges, with the most commonly occurring items listed first. It is interesting to note that low toughness material was not observed to be a primary cause of cracking in this survey, showing that specifying higher toughness will not avoid cracking problems in many cases. The most frequent type of cracking is that caused by secondary and/or distortion-induced stress. Figure 2 shows

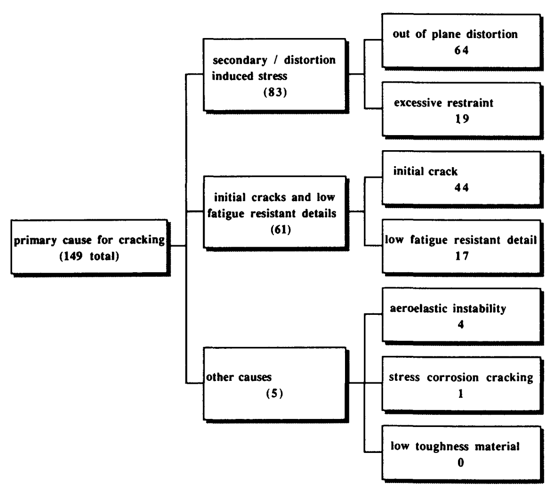

Fig. 1. Classification of primary causes for cracking in steel bridges.

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a more detailed breakdown of this major category of cracking, listing subcategories based on the kind of detail in which cracking occurs. The subclass of out-of-plane distortion in a small gap, usually a segment of a girder web, is the largest group. The local gap geometry creates an abrupt stiffness change. The effects of cyclic distortions are concentrated in this gap, leading to distortion-induced cracking. These web-gap cracks are most frequently due to distortions experienced in-service but may also be caused by handling and shipping.

2.1. Web-Gap Cracking During Service

In-service web-gap cracking is illustrated in Figure 3.[1] The floor-beam to girder connection detail shown in Fig. 3 is susceptible to distortion-induced cracking at the small web gap at the top of the floor beam connection plate. As the floor beam carries traffic loads, the end of the beam rotates, forcing the deflection of the girder web out of its normal longitudinal plane. There is a small (¾-in. to 1 in.) gap between the top of the floor beam connection plate and the top girder flange.

This web gap causes an abrupt change in stiffness, concentrating the rotation-induced distortion within a short length of the girder web. Forcing the distortion to take place in such a small space introduces high stress ranges in the gap. Repeated pumping of this short gap leads to longitudinal cracking of the web at the weld at the top of the connection plate and at the weld connecting the top flange to the web. This type of web-gap cracking at floor beam or floor beam truss connection plates is so prevalent that a survey in one state revealed cracks in half of the bridges with this detail.[6] Diaphragm and cross-beam connections are frequently the sites of similar web-gaps, but have less severe imposed rotations that the floor beam case illustrated. This kind of web-gap detail usually occurs where the top flange is in tension and arises for the ironic reason that the connection plates were not welded to the flange to eliminate an initiation site for load-induced fatigue of the tension flange.[9] To preclude distortion-induced cracking, the connection plate must be either rigidly attached to the flange as required in the AASHTO provisions, or the gap length must be significantly increased. Increasing the gap length does not always solve the problem of distortion-induced cracking. The increased flexibility may lead to increased deformations in the web gap and, consequently, the same stress levels. In-service web-gap cracking has been observed in all of the following types of details:

1. floor beam connection plates in both the positive and negative moment regions,
2. diaphragm connection plates in both the positive and negative moment regions,
3. tied-arch floor beams in the web gap at the tie girder connection, and
4. horizontal connection plates or gussets at points of lateral bracing vibration as well as at gaps between stiffeners and gussets.

2.2. Web-Gap Cracking During Shipping and Handling

Web-gap cracking happens not only under in-service conditions but also during handling and shipping. Web-gap cracking during transit is illustrated in Fig. 4.[1] The conditions present (a stiffness discontinuity, a periodic displacement, and a weld toe) are conducive to distortion-induced cracking. An abrupt change in stiffness occurs at the short web gap between the inside face of the flange and the beginning of the stiffeners. Such a gap may also occur between the inside face of the flange and the beginning of a connection plate. The periodic force operating within the gap is caused by the cyclic

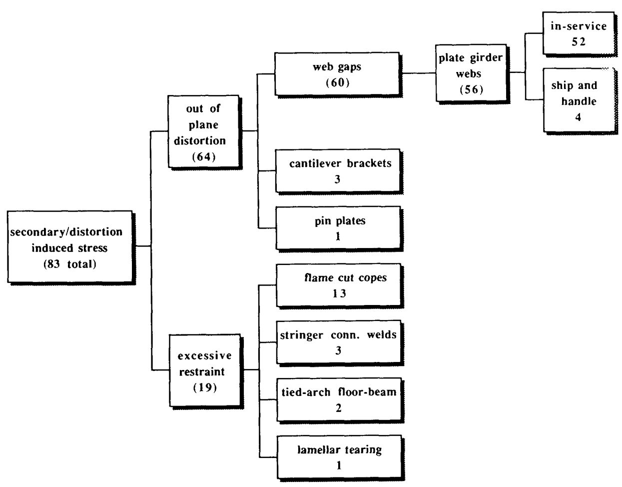

Fig. 2. Classification of secondary/distortion-induced stress cracking in steel bridges.

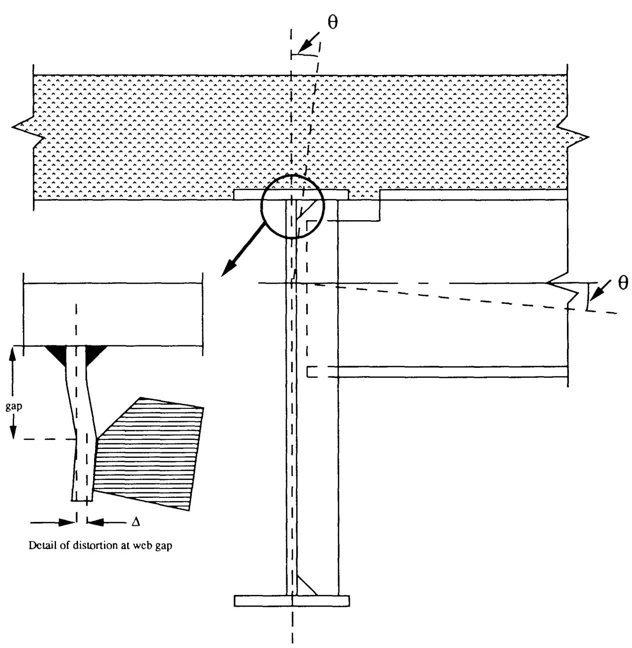

Fig. 3. Web-gap cracking during service.

distortion as the girder sways during shipment. The bottom of the girder is supported on the truck or rail car bed. The upper part of the girder displaces relative to the bottom of the girder due to the rocking motion of transit. These sway displacements can introduce large cyclic bending stresses in web-gap, leading to fatigue cracking in the web. The cracks typically begin at a weld and are oriented longitudinally on the girder, parallel to the primary axial stresses produced by in-service major-axis bending of the girder. This orientation mitigates the effect of the cracking and eases the repairs required to ensure satisfactory service behavior. Web-gap cracking has also been observed in the fabricating shop as the girders were handled and turned. Cracking in the fabrication shop also occurred in a large stiffened web plate where web-gaps existed at the intersection of vertical and transverse stiffeners that were not connected to each other.

3. CASE STUDY: DISTORTION-INDUCED CRACKING IN BUILDING TRUSSES DURING TRANSIT

Since handling and shipping practices are similar for bridge and building steel, it is not surprising that distortion-induced cracking also occurs in building components when conducive conditions are present. Distortion-induced cracking during transit is most frequently a problem for plate girders, however it may also occur in trusses, especially where thin gussets and heavy connection angles are used. This case study presents a detailed example of cracking in building trusses due to cyclic distortions during transit.

3.1. Description of Cracking

Roof trusses for an industrial plant consisted of 45 ft parallel chord trusses, 7.5 ft deep as shown in Fig. 5. The chords were structural tees. The vertical and diagonal web members were double angles. The trusses were fabricated as typical shop welded trusses, with members fillet-welded to gusset plates. The connections between the trusses and building columns were field bolted. The clip angles for these connections were shop welded to the gusset plates.

Cracks were first observed in the end gusset plates by ironworkers when unloading and sorting trusses at the erection site during the winter. An initial investigation was made at that time and concluded that the cracks were fatigue cracks caused by vibration of the cantilevered ends of the trusses during truck delivery to the site. The trusses were shipped by truck from the fabrication plant to the construction site for an over-the-road distance of approximately 800 miles. The trusses were shipped in a horizontal position as shown in Fig. 6. Trusses were stacked six high, with a weight limit of 40,000 pounds per truck. The trusses were secured with chains at both ends and at two or three panel points between. The trusses were not originally blocked at the ends but rather were shipped with the ends cantilevered from blocking at the support points, one panel point in from each end. In response to the observed cracks, changes were made to maintain minimum steel temperature during welding and to limit the vibration amplitude of the truss ends during shipping by adding end blocking as shown in Fig. 6. These changes reduced, but did not eliminate, the development of cracks. Even after the blocking changes, the top of the stacked trusses experienced

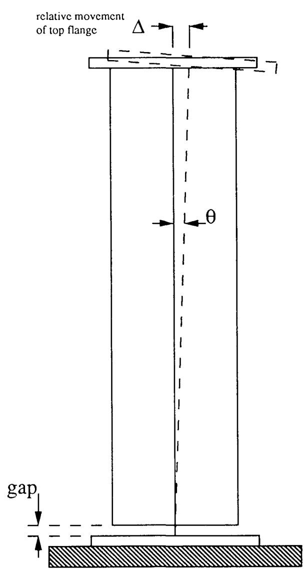

Fig. 4. Web-gap cracking during shipping.

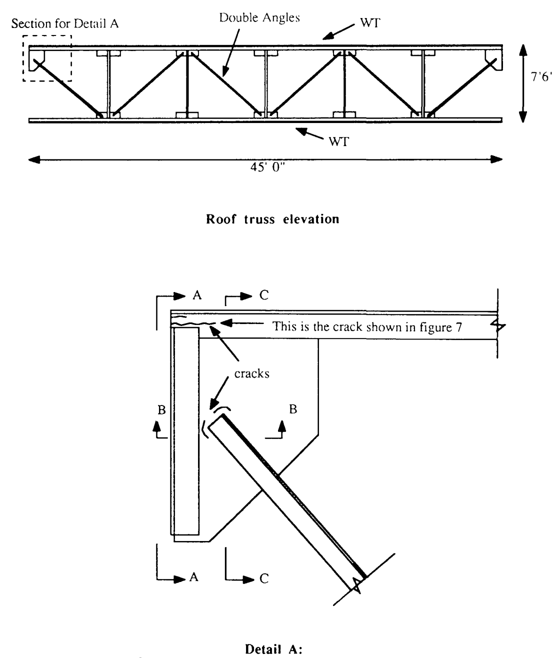

Fig. 5. Roof truss elevation and details.
Detail A: Crack locations at upper chord end gusset.
Detail B: Crack locations at bottom chord end gusset.

noticeable sway and the ends of the trusses vibrated vertically to a noticeable degree during shipping.

Cracks in the end of gusset plates again were detected visually by ironworkers while bolting end connections of trusses to columns. A visual inspection of the erected trusses identified approximately 170 trusses with cracking. Magnetic particle testing confirmed cracking in the metal for approximately 140 of these trusses. The total number of trusses on the job was approximately 4,000. All cracks were found at the ends of the trusses. No cracks were found at interior panel points. Crack lengths varied between ¼-in. and 4 in. and were typically about one inch long. Some cracks appeared on both sides of the stem or plate, while other cracks appeared on only one side. Crack locations may be categorized as follows:

1. Stem of tee top or bottom chord at base of rolled radius, running parallel to radius, see Fig. 5, Detail A.
2. Stem of tee top or bottom chord at edge of fillet weld at top of clip angle, running along weld at angle toe, see Fig. 5, Detail A.
3. Top and bottom end gusset plate at edge of fillet weld heel and toe of diagonal angles, see Fig. 5, Detail A.

Due to the persistence and extent of the cracking problem, a detailed investigation was undertaken. The objective of this investigation was to determine the probable cause of cracking. The scope of the investigation consisted of:

1. Fractographic examination of crack surfaces.
2. Fatigue analysis.
3. Probable crack cause determination.
4. Evaluation of appropriateness of repair procedures.

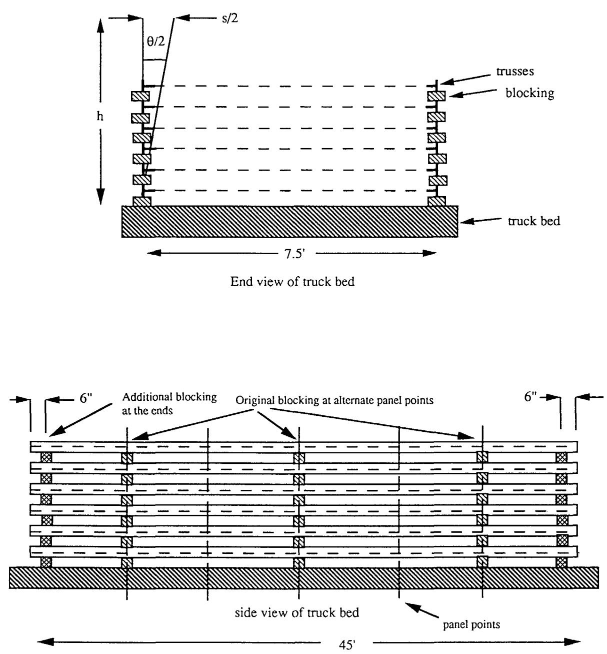

Fig. 6. Schematic of truss shipping arrangement.

The following sections discuss each of these topics and also presents material properties determined during the initial investigation.

3.2. Material Properties

Tests were conducted to establish the mechanical, chemical, and fracture properties of the steel used in the roof trusses. A standard reduced section tension test gave results of 75 ksi tensile strength, 53 ksi yield stress, and 27 percent elongation. To determine the effect of welds on these values, a reduced section tension specimen with welded bars attached gave results of 80 ksi tensile strength, 57 ksi yield stress, and 18 percent elongation. Chemical analysis showed a composition of C-0.07, Mn-0.47, Si-0.2, P-0.026, S-0.025, Ni-0.05, Cr-0.03, Mo-0.01, $C_b < 0.05$. This chemistry meets the requirements for A36 steel. Notch toughness tests were performed using half-size Charpy Vee Notch (CVN) specimens tested at 40°F with the following results:

Test #1	half-size CVN with weld	36 ft-lb
Test #1	half-size CVN without weld	18 ft-lb
Test #2	half-size CVN with weld	95 ft-lb
Test #2	half-size CVN without weld	113 ft-lb.

Notice the wide spread in results, especially for the specimens without welds. Such wide variability is indeed observable in normally specified plate steels.[11]

3.3. Fractographic Examination of the Crack Surfaces

Pieces from two cracked trusses were used to prepare crack surfaces for fractographic examination. The crack surfaces were visually examined and photographically documented, then examined using a Scanning Electron Microscope (SEM). SEM examination of all cracks matched the characteristics of known fatigue surfaces. Cracks were examined from all three types identified in Section 3.1 and shown in Fig. 5, Detail A. Figure 7 is a photograph of a crack surface located in the base metal of the stem of tee top chord, running along the fillet

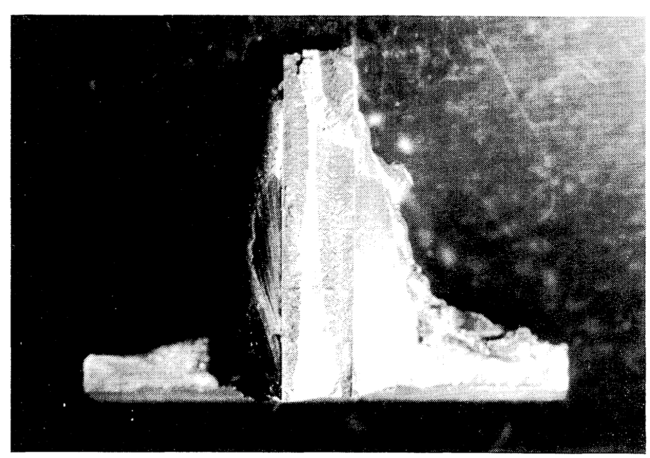

Fig. 7. Fracture surface.

weld at top of clip angle, as noted in Fig. 5, Detail A. This photograph shows the pattern typically observed where the cracks apparently initiated as edge cracks on both faces of the plate and grew to the center, meeting to create through cracks for a portion of their length.

3.4. Fatigue Analysis

3.4.1. Postulated Fatigue Mechanism

To understand the cracking behavior in these trusses, it is necessary to propose a mechanism for fatigue that agrees with the fractographic evidence and then verify by the application of fracture mechanics that known conditions would be reasonably expected to produce the observed cracks. The fatigue mechanism postulated is displacement-induced cyclic stress during shipment. The trusses were shipped by truck in a horizontal position stacked six trusses high as shown in Fig. 6. The top of the stacked trusses swayed and the ends of the trusses vibrated to an extent easily observable by the eye during shipping. Distortion-induced stress would have occurred during transit due to differential rotation across small gaps. The magnitude of the stress range for a particular gap would have been dependant on the amount of rotation occurring across that gap. Gaps and associated rotations are illustrated for all three cracking types identified in Section 3.1 and shown in Fig. 5, Detail A. The size of the gaps for Section A-A and B-B is approximately 1.5 in. The gap in Section C-C is much larger, measuring about one foot. Section A-A marked on Fig. 5, Detail A is drawn in Fig. 8. This section shows how a cyclic vibration at the end of the clip angle leads to a stress range in the gap in the stem of the tee chord between the rolled radius and the fillet at the top of the clip angle. The stress range for the gusset plate at the gap between the toe of the diagonal and the toe of the clip angle on the Section B-B shown in Fig. 5, Detail A may be addressed in a similar way. Section C-C shown in Fig. 5, Detail A, experiences rotation due to the sway of the stacked trusses as shown in Fig. 6.

3.4.2. Estimate of Stress Range and Number of Cycles

To perform a fatigue analysis, information is needed about the magnitude of the stresses and the number of stress cycles during shipment. Since shipment was complete, this data was not directly available. Reasonable estimates were computed as follows. The stress range for full stress reversals, as seen in this case, is equal to twice the maximum stress. The vibration amplitude is equal to twice the amount of the displacement to one side. The vibration amplitudes needed to produce the maximum possible stress ranges of twice the yield stress were calculated to be:

d = displacement at end of clip angle (Fig. 8)	0.4 in.
s = sway at top of stacked trusses (Fig. 6)	4.5 in.

Therefore, it is reasonable to assume that the stress ranges experienced at the observed locations could have been as high as twice the yield stress.

The number of stress cycles may be bracketed by upper and lower limits as follows. The natural frequency of the clip angle extension (Fig. 8) was observed to be about eight cycles per second. At a speed of 50 miles per hour, this gives 567 cycles per mile which compares reasonably to known shipping cycles of 500 cycles per miles for rail.[1] For a shipping distance of 800 miles, this results in 450,000 cycles or approximately half a million cycles as a reasonable upper bound. The sway frequency of the stacked trusses may be roughly guessed at less than one cycle per second. The lower bound may be set at a order of magnitude less than the upper, or approximately 45,000 cycles.

3.4.3. Crack Growth by Fatigue

Using the principles of fracture mechanics, a fatigue analysis was performed to determine what stress ranges would be required to produce the observed cracking for the high and low estimates of stress cycles during shipping. The crack configurations analyzed were one and two sided edge cracks in a finite width plate subject to in-plane bending.[10] Normal initial flaw sizes of 0.005 in. for rolled sections[12] and 0.03 in. for fillet welds[1] were assumed for analysis. The initiation stage thus was assumed not to contribute to the fatigue life. The entire fatigue life was modeled by the propagation stage. An appropriate computer program was used to model the cracks.[8] Results of this analysis are summarized below.

Stress range (ksi) required to propagate crack through plate for high estimate of 450,000 stress cycles:

	one-sided crack	*two-sided crack*
Stem of WT at rolled radius	28	25
Stem of WT at weld	18	12
Gusset plate at weld	18	13

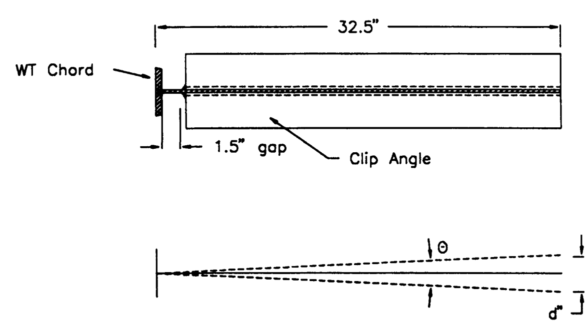

Fig. 8. Deformation in stem of tee, Section A-A from Fig. 5 Detail A.

Stress range (ksi) required to propagate crack through plate for low estimate of 45,000 stress cycles:

	one-sided crack	*two-sided crack*
Stem of WT at rolled radius	58	54
Stem of WT at weld	38	26
Gusset plate at weld	38	28

These stress ranges correspond to vibration amplitudes of 0.04 in. (12 ksi) to 0.2 in. (58 ksi) at the end of the clip angle and 0.5 in. (12 ksi) to 2.5 in. (58 ksi) for sway at top of stacked trusses. These figures are well within expected deflection ranges for trusses shipped in this manner.

3.4.4. Fracture Toughness

The fractographic examination showed crack growth by fatigue with no indication of fast fracture. The fatigue analysis results had a maximum stress intensity factor, K, of 20.7 ksi$\sqrt{\text{in.}}$ The material test results showed a minimum CVN of 18 ft-lb at 40°F. Using the two-stage CVN-K_{Id}-K_{Ic} correlation[13] on the minimum CVN value gives a fracture toughness of 51 ksi$\sqrt{\text{in.}}$ for a dynamic strain rate ($\dot{\varepsilon} \approx 10^{-1}$ sec^{-1}) at a temperature of 40°F and a fracture toughness of 51 ksi$\sqrt{\text{in.}}$ for an intermediate strain rate ($\dot{\varepsilon} \approx 10^{-3}$ sec^{-1}) at a temperature of −62°F. This is in agreement with the observed behavior of crack growth only by fatigue for winter truck shipment.

3.5. Probable Cause and Sequence of Crack Growth

The probable cause of cracking was therefore concluded to be displacement-induced cyclic stress during shipment. The observed crack locations were in thin plates subject to cyclic bending where short gaps would be expected to result in a geometric amplification of the cyclic stress leading to cracking. The cracks initiated on one or both faces of the plates, most frequently at the toe of fillet welds but sometimes at a rolled radius. The cracks grew by fatigue through the thickness of the plates. Vibration amplitudes of 0.04 in. to 0.2 in. at the end of the clip angle and 0.5 in. to 2.5 in. for sway at the top of stacked trusses are sufficient to produce the observed cracking over the given shipment distance. The vibration amplitudes and number of loading cycles required were in agreement with expected ranges for trusses shipped in this manner. For this case, blocking of the clip angle (Fig. 5, Detail A) would be recommended for proper shipping.

3.6. Evaluation of Appropriateness of Repair Procedures

The cracks in the roof trusses were repaired by gouging and rewelding. Although this was an appropriate repair technique for building trusses subject to static loads, gouging and rewelding is not an appropriate repair for bridge members. An alternate suggested repair of drilling holes approximately ½-in. in diameter at the ends of the cracks was not used. Either of these repair procedures were appropriate in view of the cause of cracking and the required in-service behavior of the trusses because further fatigue crack propagation during service loading would not be expected.

The cracks in the stem of the tee at the rolled radius and at the top of the clip angle were in regions carrying negligible loads in service. This material could be completely coped out without decreasing the strength of the trusses for in-plane behavior. Gouging and rewelding was a more than adequate fix in this region. Drilling holes at the end of the cracks to eliminate the stress concentration at the sharp crack tip would also have been a satisfactory fix.

The cracks at the ends of the diagonals were in a stressed region of the gusset plate. Gouging and rewelding was an adequate fix in this region. Where sufficient net area and sound weld remained for full load transfer, drilling holes at the ends of the cracks also would have provided a satisfactory repair at the ends of the diagonals. The condition of the crack with end holes would have then been analogous to any other situation resulting in a reduced section at this location such as the presence of bolt holes or a penetration opening through the gusset.

In summary, the cracks were caused by out-of-plane dynamic vibrations during shipment. The in-service trusses carry in-plane static loads. The cyclic stresses leading to fatigue crack growth are not present under service conditions and repaired cracks would not be expected to re-initiate.

In this case study, the fatigue cracks were detected and repairs performed. It is of interest to postulate the case where the cracks were not detected and the resulting implications for in-service performance of the cracked trusses. As mentioned above, the cracks in the stem of the tee and at the top of the clip angle were essentially unstressed. The cracks at the ends of the diagonals did not reduce the section enough to govern the load carrying capacity. The cracks would not grow by fatigue in-service, since the loads would be static. The existing cracks would not become unstable, under design loads, so fracture failure would not occur. Thus, for this particular case, the trusses would be expected to perform satisfactorily in-service even if the cracks were undetected.

4. CONCLUSIONS AND RECOMMENDATIONS

4.1. Conditions that Lead to Cracking During Shipment

Two conditions must coexist to cause distortion-induced cracking:

1. an abrupt change in stiffness, and
2. a recurring displacement taking place across this stiffness discontinuity.

In addition, presence of a weld toe within this small region acts as a crack initiation site and exacerbates the problem. These conditions conducive to cracking occur most frequently in plate girders, especially where there are small web gaps. The conditions may also arise for trusses, especially where thin plates and heavy angles cause severe stiffness changes in the gussets. The extent of cracking will depend on the specific gap configuration, the magnitude of the stress ranges induced by the distortions, and the number of fatigue cycles. Particular connection details, plate thicknesses, and displacements, as well as length of trip, thus all play a role.

4.2. Effect of Cracking in Transit on Structural Performance

When cracking occurs due to a low fatigue resistant detail or a large initial defect, only one or a few significant cracks are usually generated. Observation of these cracks allows action to be taken before cracking occurs at many points in the structure. Unlike this load-induced crack scenario, distortion-induced cracks frequently form at the same time in many locations. This means that many cracks must be repaired. For cracking that occurs during transit instead of during service, it may be possible to restrict the number of cracks, depending on the type of shipment. If a large number of components are shipped in a similar fashion at the same time by rail, the possibility exists for many cracks to form simultaneously. For components shipped by truck, careful examination of components from the first truckloads shipped can provide a warning so that relatively few cracks occur.

Distortion-induced cracking during transit is usually due to out-of-plane movement. Since components are designed for in-plane behavior, the cracks usually form parallel to the design tensile stresses. Such cracks running parallel to the in-service stresses may not be harmful to the structure's performance as long as they are identified and repaired before they turn perpendicular to the in-service stresses. Repair of these cracks is frequently straight forward, especially for building components where cyclic stress are not a major characteristic of in-service loads. The simplest approach is to drill out the crack tips and check the capacity of the reduced section to carry design loads. Gouging and rewelding is another alternative, for building components where cracks are located in regions of low in-service stress and not subject to cyclic stresses, so that repaired cracks are unlikely to re-initiate. Gouging and rewelding is not an appropriate repair for bridge members. More care must be taken in the case of repair of cracked bridge components and case studies are available in the literature.[1,9,5]

4.3. Prevention of Distortion-Induced Cracking During Transit

When designing connection details and attachments such as stiffeners, attention should be paid to how the member will be shipped. Avoid creating short gaps which will be subjected to differential movement across the gap. If such a gap must occur, the severity of the stiffness change may be smoothed out by elongating the gap, making the material in the gap stiffer (usually by increasing its thickness) and/or making the material bounding the gap more flexible (usually by decreasing its thickness). Loads should be arranged and blocked properly to prevent distortions in gaps. Following good practice by careful inspection of components from the first truckloads shipped at the shop and at the site will rapidly identify transit cracking and avoid costly rejection or rework on many components. Implementation of these recommendations should minimize the possibility of distortion-induced cracking during transit.

REFERENCES

1. Fisher, John W., *Fatigue and Fracture in Steel Bridges: Case Studies,* John Wiley & Sons, New York, 1984.
2. American Association of State Highway Officials, *Guide Specifications for Fatigue Design of Steel Bridges,* Washington, D.C., 1989.
3. AISC, *Manual of Steel Construction, Allowable Stress Design,* 1st ed. Chicago, IL, American Institute of Steel Construction, 1989.
4. AISC, *Manual of Steel Construction, Load and Resistance Factor Design,* 9th ed. Chicago, IL, American Institute of Steel Construction, 1986.
5. Kulicki, John M., and Dennis R. Mertz, *Case Studies of Displacement-Induced Fatigue,* Sixth Annual Structures Congress, American Society of Civil Engineers, 1987.
6. Fisher, John. W., and Umur Yuceoglu, *Fatigue and Fracture in Steel Bridges: Case Studies,* Fritz Engineering Laboratory Report 448-2(81), Lehigh University, Bethlehem, PA, 1981.
7. Demers, Cornelia E. and John W. Fisher, *Fatigue Cracking of Steel Bridge Structures, Volume I: A Survey of Localized Cracking in Steel Bridges—1981 to 1988,* FHWA-RD-89-166, Federal Highway Administration, McLean, VA, 1990.
8. Roddis, W. M. Kim, *Heuristic, Qualitative, and Quantitative Reasoning About Steel Bridge Fatigue and Fracture,* Ph.D. Thesis, Civil Engineering Department, Massachusetts Institute of Technology, Cambridge, MA, 1989.
9. Fisher, John W., and Dennis. R. Mertz, *Retrofitting Steel Bridges to Extend Their Fatigue Lives,* The 1985 International Engineering Symposium on Structural Steel, American Institute of Steel Construction, Chicago, IL, 1985.
10. Tada, H., P. C. Paris, and G. R. Irwin, *The Stress Analysis of Cracks Handbook,* 2nd ed., Paris Productions, Inc., 226 Woodbourne Dr., St. Louis, MO, 1985.
11. AISI Technical Committee on Plates and Shapes, *The Variations of Charpy V-Notch Impact Test Properties in Steel Plates,* American Iron and Steel Institute, Washington, D.C., 1979.
12. Fisher, J. W., K. H. Frank. M. A. Hirt, and B. M. McNamee, *Effect of Weldments on the Fatigue Strength of Steel Beams,* National Cooperative Highway Research Program Report 102, Transportation Research Board, Washington, D.C., 1970.
13. Barsom, John M., and Stanley T. Rolfe, *Fracture and Fatigue Control in Structures,* Prentice-Hall, Englewood Cliffs, New Jersey, 1987.