Zeitschrift: IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht

Band: 11 (1980)

Artikel: Fracture and retrofit of Dan Ryan Rapid Transit Structure

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DOI: https://doi.org/10.5169/seals-11376

IX

Fracture and Retrofit of Dan Ryan Rapid Transit Structure

Fissuration et réparation du pont du métro Dan Ryan

Sprödbruch und Abänderung der Dan Ryan Hochbahn Brücke

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SUMMARY
Three box frame bents of the Chicago Rapid Transit elevated structure cracked during a cold period in 1978. Fatigue crack growth developed at the intersection of the girders and bent side plates where large crack of fusion areas and high stress concentration existed. Brittle fracture followed. An analysis of the different crack growth stages and the retrofitting methods are provided.

RESUME
Trois poutres caisson des cadres transversaux du métro de Chicago se sont fissurées à une période de basse température pendant l'hiver 1978. Les fissures de fatigue se sont formées aux intersections des longerons et des âmes des poutres caisson, dans des zones ayant des contraintes résiduelles élévées. La rupture fragile en est la conséquence. Les différents stades de fissuration et les méthodes de réparation sont présentés dans le rapport.

ZUSAMMENFASSUNG
Drei kastenförmige Rahmen einer Hochbahnstrecke in Chicago brachen während des kalten Winters 1978. Ermüdungsrisse starteten in den Schweissverbindungen zwischen den Längsträgern und den Stegblechen des Kastenträgers, in einer Region mit hohen Eigenspannungen. Sprödbruch folgte. Die verschiedenen Risswachstumsstadien und Sanierungsmassnahmen werden beschrieben.

1. Introduction

A large crack in one of the steel box bents (rigid frames) supporting the elevated track of Chicago's Mass Transit System (EL) of the Dan Ryan Line was discovered on January 4, 1978 [1]. Subsequent inspection showed that two adjacent bents were also cracked. The structure was immediately closed to traffic and was reopened to traffic on January 15, 1978, after shoring towers were completed. During the two weeks prior to discovery of the cracks low temperatures of -20° C were recorded in Chicago.

The fractured bents are part of a 12 m high 820 m long viaduct. They support four articulated plate girders which carry the cast-in-place concrete deck. The rails of the two tracks are on wooden ties resting on ballast. The tracks and the structure are on a 120 m radius curve at the location of the fractured bents.

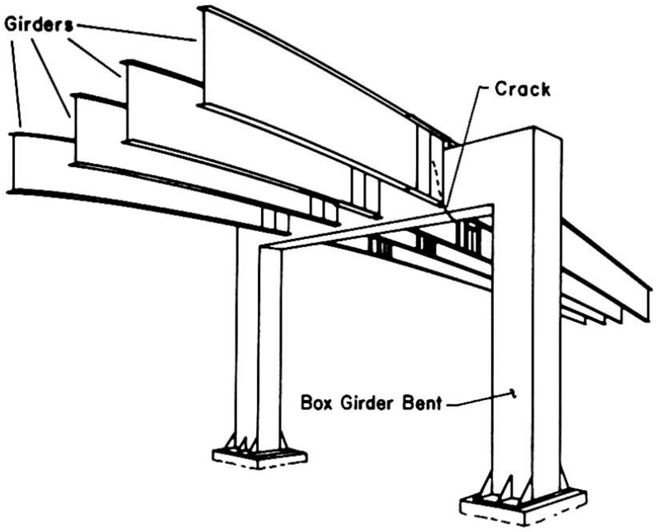

Fig. 1 Schematic Showing Box-Shaped Bent and Girders with Crack Location

The bents have a box-shaped cross-section consisting of two column legs and a horizontal box member, as shown in Fig. 1. The top flanges of the plate girders pass over the top flanges of the boxes. The bottom flanges pierce the boxes through flame-cut slots near the bottom of the box side plates. In the fractured bents, the intersections were connected by heavy fillet welds on the outside and little or no weld on the inside. At other locations, the intersections were sometimes filled with welds. The girders intersect the boxes at different angles. The bents and the girders are fabricated of American Society for Testing of Materials (ASTM) A36 steel.

The structure was designed in 1967 in accord with criteria and procedures established by the American Railway Engineering Association (AREA) and built in 1968-1969. Appearance of the structure was an important consideration and this type of construction minimizes the height of the structural system.

Since traffic operations began, the structure was inspected several times. The last inspection before the detection of the cracks was in July 1976.

The initial field examination of the fracture indicated that all of the cracks started at the welded junction of the plate girder flange tip to the box side plate, as illustrated in Fig. 2. All three cracks completely severed the bottom flange of the box girders and the webs. Openings at the bottom flange of about 20 mm were measured.

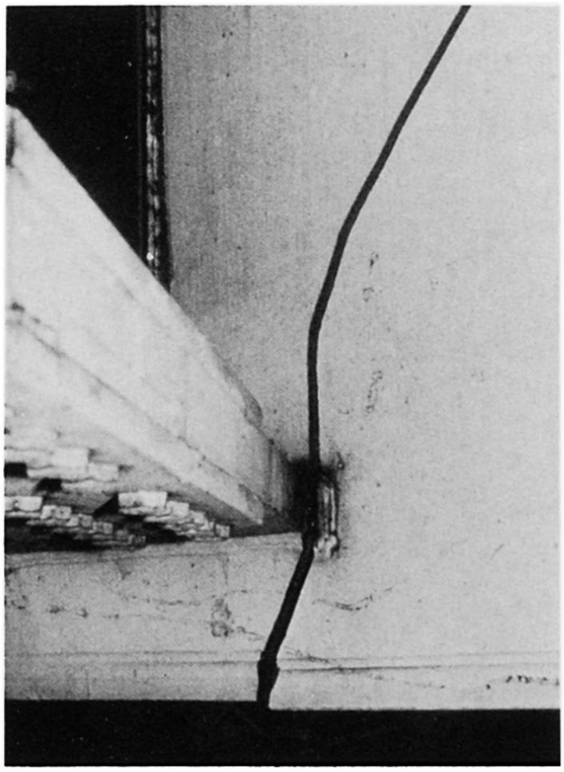

Fig. 2 Cracked Box Side Plate

The cracks were arrested near or in the top flange. The cracks in the bottom flanges were inclined approximately 20 degrees to a normal to the longitudinal axis of the bent above the flanges of the girders. Two crack surfaces were found to be slightly corroded, the third one had a heavy oxide coating.

2. Material Properties, Stresses at the Critical Location

Samples were removed from the box girders for laboratory investigations. From this material, specimens for tensile tests, chemical analysis, Charpy V-Notch tests, compact tension fracture tests, and metallographic and fractographic examinations were made. The tests were carried out in accordance with standard ASTM Specifications at several laboratories. The physical and chemical tests indicated that the material conformed to the ASTM requirements for A36 steel.

The yield strength (0.2% offset) measured with flat tensile specimens was determined to be 235 MPa and the tensile strength to be 480 MPa. The Charpy V-Notch tests showed good toughness with impact energies greater than the American Association of State Highway and Transportation Officials (AASHTO) minimum requirement of 20 joules at 4° C, as specified for a minimum service temperature down to -34° C for Zone 2. The transition temperature is between 4° C and -20° C for 20 joules.

Compact tension fracture specimens were tested at one second loading rate at -1° C to -34° C. The tests did not fulfill the requirements of the ASTM E399 specification, and the fracture toughness had to be estimated using the J-integral method. A critical plane strain material toughness K_{Ic} (average of three specimens) of 88 MPa $\sqrt{m}$ was measured at -20° C.

A review of the stresses was made using standard methods of analysis [1]. It was found that the magnitude of the tensile and compression stresses due to the dead load, design live load, impact and centrifugal forces were on the order of 120 MPa, and the shear stresses about 105 MPa. The calculated stresses were within the allowable limits of the design specifications. At the location of the fracture where the heavy oxide coating was observed, the computed shear stress was about 20 percent higher than the allowable stress.

The tensile stress range in the box side plates at the tip of the girder flange framing into the box girder (location of the crack) was calculated to be 22.2 MPa for the design live load (both tracks loaded), including impact and centrifugal forces. On an average weekday, 467 trains (in both directions) pass over this viaduct. The trains vary in length between two and eight cars. Most stress cycles during service are produced by one train crossing over the structure at a time. In addition, the impact factor will likely be smaller than the design value. Hence, the service stress cycles from a train probably varied between 7 MPa and 10 MPa.

3. Analysis of the Fracture

The crack surfaces of the side plates of the box girders adjacent to the flange tip were examined. Chevron markings found on the fracture surface indicated that the crack originated near the tip of the girder flange. A photograph and schematic of one fracture surface are shown in Figs. 3a and 3b. The overall fracture surface shows that the weldment connecting the flange tip to the box had a large lack of fusion area. Paint was found in some of these areas, which had penetrated from the inside of the box. The largest lack of fusion area was approximately 18 mm by 83 mm and was approximately elliptical shaped.

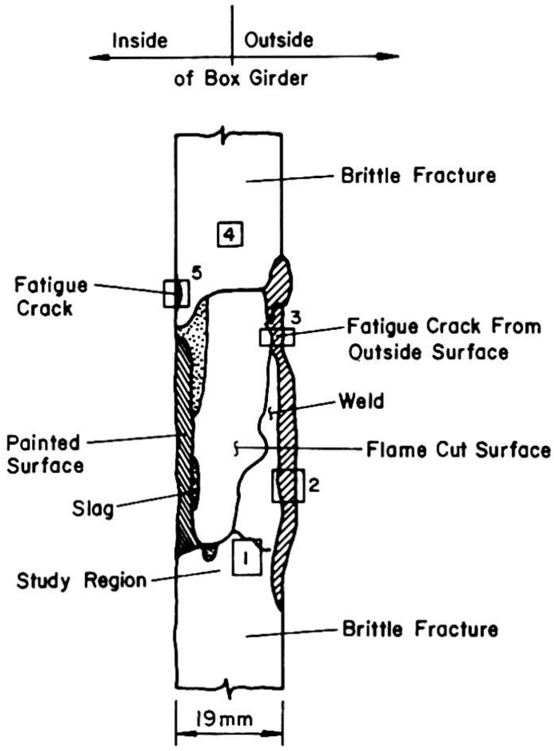

Fig. 3b Schematic of Fracture Surface

Fig. 3a Fracture Surface of Box Girder Web Plate

Surface replicas were prepared from different locations of the crack surface indicated in Fig. 3b and investigated with the transmission electron microscope. Fatigue striations (see Fig. 4) were found on all three fracture surfaces. These striations represent the extension of the crack front during one load cycle and confirmed the development of fatigue crack growth. Striation markings were found in regions 2 and 3 in Fig. 3b; the measured growth rate was 2.10^{-5} mm/cycle near the exterior surface and 5.10^{-5} mm/cycle near the lack of fusion area. The examination of region 4 near the upper end of the discontinuity showed mainly cleavage fracture with river patterns. The examination of each fracture surface confirmed that fatigue crack growth originated at the lack of fusion area and at the exterior weld surface at the weld toe on the box side plate. It was estimated that about 6 mm of fatigue crack growth was experienced before the cracks became unstable.

The stress intensity range ΔK can be estimated from the stress range $\Delta\sigma$ as [2]

$$\Delta K = F_s \cdot F_w \cdot F_e \cdot F_g \cdot \Delta\sigma\,(\pi a)^{1/2} \quad (1)$$

where F_s is the front free surface correction factor, F_w is the back free surface correction factor, F_e is the crack shape correction factor and F_g is the stress gradient correction factor. "a" is the crack size. For the geometrical

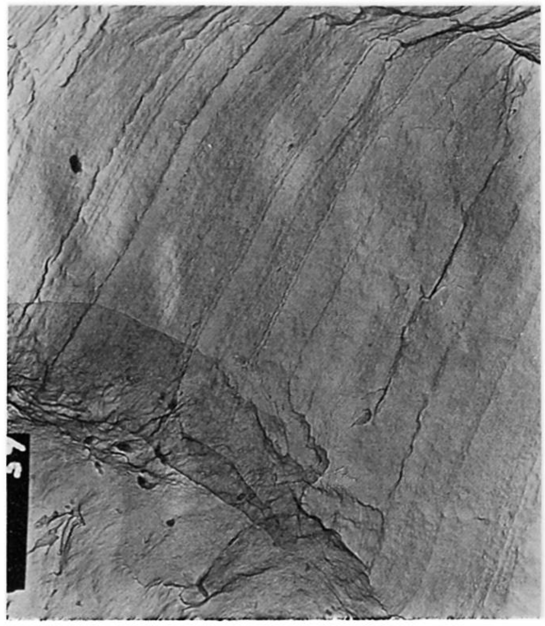

Fig. 4 Fatigue Striation Markings on Box Girder Web Surface

conditions that existed at this detail, ΔK was estimated to be between 0.38 $\Delta\sigma$ and 0.45 $\Delta\sigma$ MPa $\sqrt{m}$. Stress ranges between 6 MPa and 10 MPa would exceed the crack growth threshold which is about 2.9 MPa $\sqrt{m}$. The stress intensity factor estimated from the striation markings is in reasonable agreement with these results.

The stress intensity factor for a crack growing at the weld toe of the exterior weld surface is about 0.30 $\Delta\sigma$ MPa $\sqrt{m}$ when the stress gradient correction F_g = 3.16. Hence fatigue crack growth would be expected at the weld toe as well.

All three bents experienced cleavage or brittle fracture after the fatigue crack had propagated through the weld metal and resulted in a through crack at the stringer flange tip. The small ligaments which remained between the lack of fusion region and the external fatigue cracks were ignored; this results in a stress intensity factor estimate of [3]:

$$K = \sigma\ (\pi a)^{1/2} \tag{2}$$

For a crack length of 2a = 0.1 m, the K value from Eq. 2 is 0.40 σ MPa $\sqrt{m}$. The K value was also calculated assuming a semielliptical surface crack with appropriate correction factors [3]. This results in a stress intensity factor of 0.29 σ MPa $\sqrt{m}$.

Since welding around the flange tip produces a high residual tensile stress field, the critical stress at the crack tip can be assumed to be at the yield point. At room temperature the yield point is 235 MPa. Taking into account the low temperature at the time of fracture, the yield point was estimated to be 300 MPa. Hence, the estimated maximum stress intensity at the fatigue crack tip was between 87 MPa $\sqrt{m}$ and 120 MPa $\sqrt{m}$.

The fracture toughness at -20° C and a one second loading rate was found to be 88 MPa $\sqrt{m}$. Hence, brittle fracture was a reasonable development after the fatigue crack growth that was experienced during the eight years of service.

After the discovery of the fracture in the Dan Ryan Rapid Transit, fatigue tests of similar details were carried out at Fritz Engineering Laboratory, Lehigh University [4]. It was found that the fatigue strength of a web detail which simulated the penetration of a girder flange through the web was well below the most severe design category in the AASHTO Specifications [5]. Figure 5 shows the fatigue resistance provided by the tests of the simulated web penetration insert detail [4].

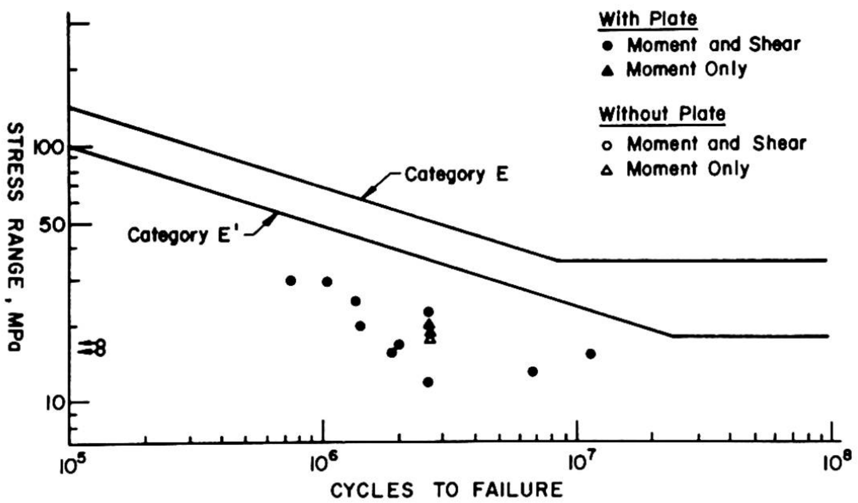

Fig. 5 Fatigue Strength of Simulated Web Penetration

4. Retrofitting Other Details

The cracking of three bents in the viaduct of the Dan Ryan line developed from fatigue crack growth at the weld junctions of girders which pierced the side plates of the box-shaped bents. Subsequent inspection revealed fatigue cracks at similar locations in nearby bents. Brittle fracture occurred when a critical combination of crack size and stress resulted in a critical stress intensity equal to the material fracture toughness at the low temperature. The cracks developed in the bents at the junction of the girder flange and side plates. The initial fabrication conditions provided cracklike conditions that were very susceptible to fatigue crack growth. It was a difficult area to weld because of the skewed angle between the girders and the bent. Comparable conditions have been observed in other structures with similar details and steps are being taken to retrofit those structures as well.

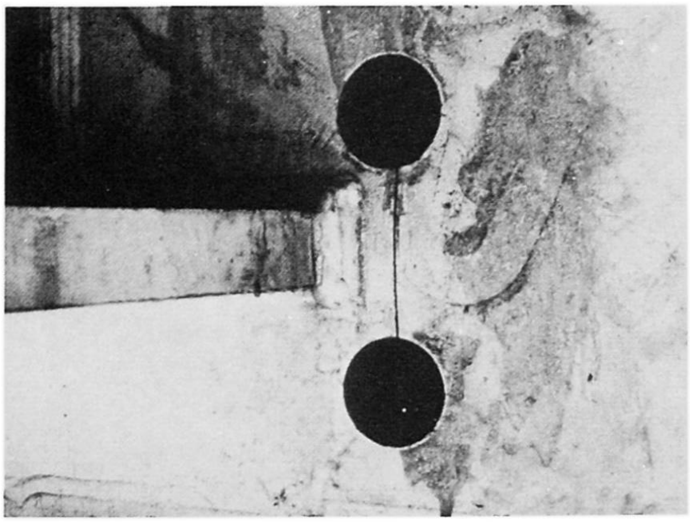

Fig. 6a Retrofitted Box Side Plate

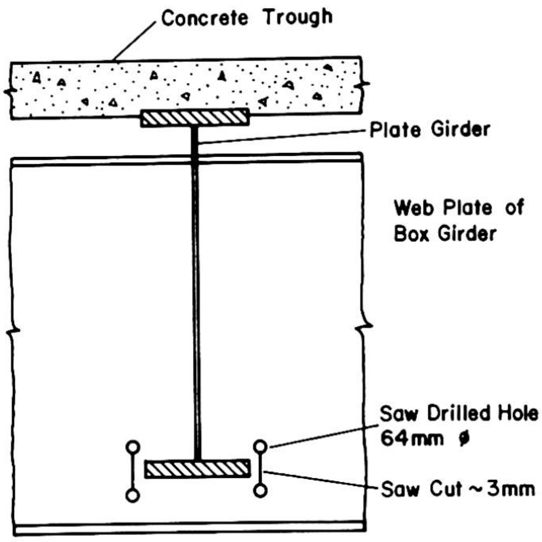

Fig. 6b Schematic of Retrofitted Detail

Any retrofit must minimize the severe stress concentration at the intersection of the girder flange and the box girder webs and reduce the high residual stresses from the weld shrinkage. To accomplish this aim, those locations with small or negligible cracks at the flange tips were retrofitted by cutting holes and sawing between them to create a "dumbbell"configuration in the box side plates near the edges of the girder flanges, as shown in Fig. 6. This method of retrofitting was considered to release the residual stress field at the flange tip and to shield the critical location from the bending stresses.

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