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Some Remarks on the Service Behaviour of Steel Railway Bridges

Quelques remarques sur le comportement en service de ponts en acier

Einige Bemerkungen über die Bewährung von Eisenbahnbrücken aus Stahl

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SUMMARY
The paper touches on the assessment of the actual behaviour of several bridges with emphasis on load spectra, actual field measured behaviour, fatigue and fracture behaviour, bridge fires and the railway's control plans for the above. The reduction in safety against brittle fracture in changing from riveted to welded structures is pointed out.

RESUME
L'article traite de l'évaluation du comportement de divers ponts et spécialement du spectre des charges du trafic, de la performance réelle mesurée sur l'ouvrage, du comportement à la rupture fragile et à la fatigue, des incendies des ponts, et les mesures de contrôle adoptées par la Compagnie. L'article décrit également la diminution du coefficient de sécurité à la rupture fragile d'une structure soudée comparée à une structure rivetée.

ZUSAMMENFASSUNG
Der Artikel behandelt das Verhalten mehrerer Brücken: Lastspektren, Messungen am Bauwerk, Verhalten während der Ermüdungszeit und des Bruches, Feuer an Brücken und Schadenverhütungsmassnahmen an Eisenbahnbrücken. Es wird unter anderem beschrieben, wie der Wechsel von genieteten zu geschweissten Brücken zu einer Verminderung der Sicherheit gegen Bruchfestigkeit im spröden Bereich geführt hat.

1. INTRODUCTION

With over 3,500 steel bridges varying in size from the largest cantilever bridge in the world [1] to some very small structures, a similar number of timber structures, and a smaller amount of reinforced and prestressed concrete bridges, CN Rail is in a good position to observe the service behaviour of bridges in the northern part of North America.

The average age of CN's steel bridges is over 60 years with the oldest surviving rail carrying super-structure being 96 years old. Because of generous safety factors and good detailing, CN steel bridges have and continue to serve the Railway well. Our current design loading is Cooper's E-80 as per the American Railway Engineering Association (AREA) Manual [2].

Typical current loadings are in the Cooper's E-50 to E-60 range, typified by the 91 metric tonne (100 ton) capacity car weighing 119.3 tonnes (263,000 lbs.) gross on four axles with an equivalent load per foot of track of not more than 8.93 tonnes per meter (6,000 plf) from coupler to coupler, and six axle locomotives weighing 176.4 tonnes (389,000 lbs.) and meeting the same criteria. There are of course occasional heavy loads. The heaviest the author has cleared was equivalent to Cooper's E-109 or roughly double the standard heavy car. There are, in addition, several lines carrying captive ore cars with Cooper's equivalent ratings in the E-60 to E-80 class.

The distribution of current bridge service loadings (Load Spectra) is quite wide as shown in figure 1. The distribution shown in Figure 1(a) represents a line carrying primarily bulk commodities and some small quantities of mixed freight to the west coast port of Vancouver, B.C. [3]. The distribution shown in Figure 1(b) represents a good mix of freight with a predominance of grain traffic just west of Winnipeg, Manitoba, roughly in the middle of Canada. The distribution shown in Figure 1(c) represents an ore carrying line leading from Northern Ontario. Finally, the distribution shown in Figure 1(d), just west of Montreal, shows the influence of relatively dense passenger traffic on the load spectrum.

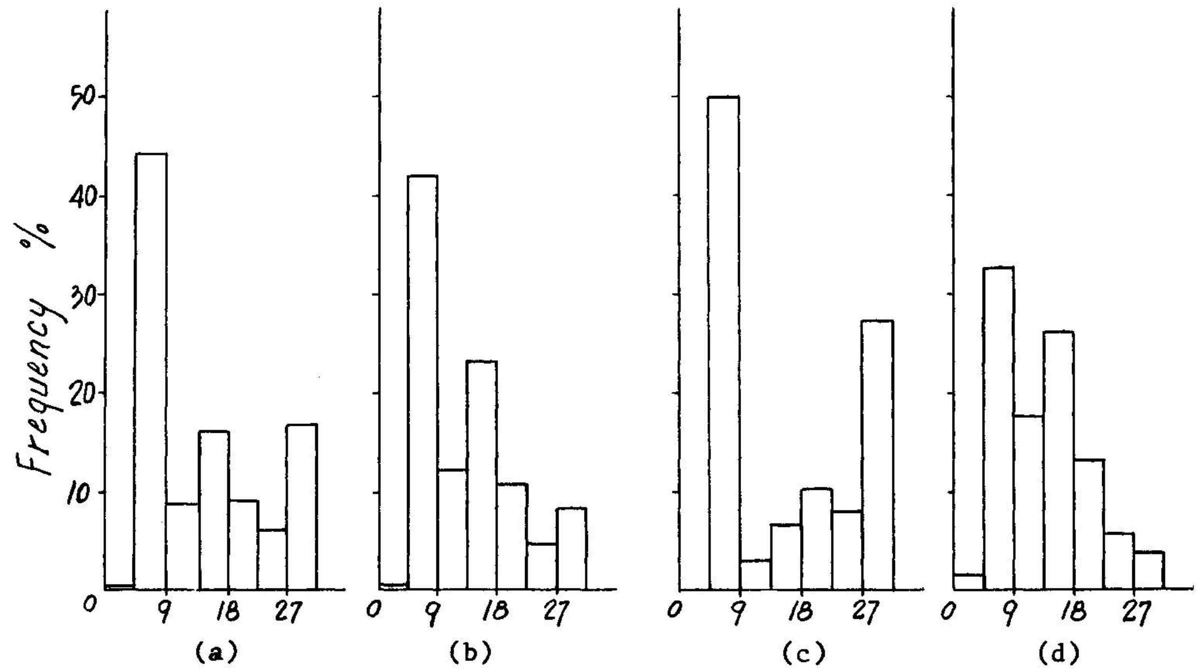

Fig. 1 Axle Load Histograms (Metric Tonnes)

As time goes on and our Railway becomes more efficient, the percentage of heavy traffic will increase [4]. On short spans, this will accentuate fatigue problems on those spans where these loadings were not adequately considered.

Six years ago, we examined our main line steel bridges and found 30 or so about which we were concerned. Several were replaced as it was less expensive to do so than to subject them to detailed analysis. Others were eliminated from consideration because redundancy would prevent collapse when cracking occurred. Of the remainder, six were chosen for detailed study. The emphasis has been placed on either major or typical structures where normal inspection would be unlikely to either find a crack or would not detect it in time. In short, those structures with little or no fail safe capability.

In examining the variables which control fatigue failure, the three most significant are geometry of the detail, stress range and load spectrum.

The influence of detail geometry, primarily stress raisers and defects have been studied by others [5]. Laboratory studies can be used to predict the useful life of various details with a degree of statistical certainty. On some details where insufficient information was available, we conducted our own tests as in the case of the fatigue life of wrought iron beams with holes in the bottom flange. One of the spans we were examining was destroyed by a barge just as a report was being prepared on its potential for fatigue failure. The pin plates were salvaged and examined to verify the bearing angle, size of initial defect and so on.

Nevertheless, critical details can usually be determined from the drawings supplemented by a field inspection. A preliminary check can be made to see if there is a potential problem. If there is, then a more detailed analysis may be necessary.

A rough idea of the loads to which a structure has been subjected can be gleaned from records of the loads carried. This has meant hours of laborious cross checking of data and of extracting from hand written records going back to the time before the formation of I.A.B.S.E. as computerized records were only begun in 1968. Fortunately, light cars tend to do little damage and ignoring them induces little if any error. It is indeed fortunate that the Miner-Palmgren Law (n/N) supports this. History also makes the task easier (or perhaps possible) in most cases. With the exception of locomotives, heavy vehicles in regular service are a recent phenomena. On our railroad, 91 tonne capacity, 4 axle cars (119.3 metric tonnes gross to rail) began to appear in regular service in the early 1960's. The introduction of major main line long hauls of bulk commodities started in April 1970.

To illustrate the point, we need only consider the gross ton miles carried by the Fraser River Bridge (F.R.B.) at New Westminster, B.C. One can see the tremendous increase in traffic that started in the late 1960's (Figure 2). It is evident that the traffic in this and the next decade will have a predominant bearing on the fatigue life of this bridge.

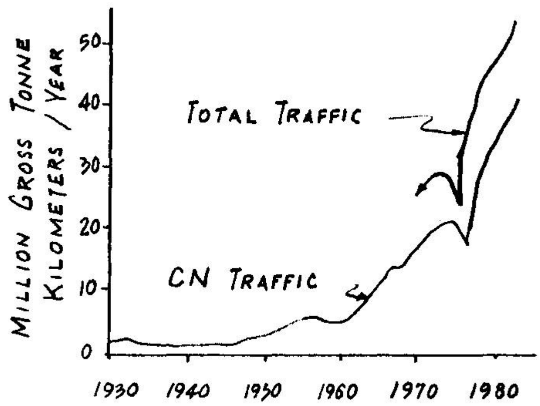

Fig. 2 Annual Tonnage (F.R.B.)

A sample of regular traffic was weighed at speed at each of the six locations under consideration using rail shear circuits developed by our own laboratory. This enabled us to check the effect of sequence which on the bridges examined so far has proven to be of no significance, and to verify the accuracy of the computerized data base being used to record the analytical load spectrum. Correlation has been quite good after correcting for known dynamic effects.

Recently, some concern [6] has been caused about the phenomena of high frequency vibrations in certain members. This is being examined.

Knowing the load, one then requires the stress that it causes at each critical detail. Load paths in some structures are too complicated to precisely determine the stress range by calculation alone. The actual behavior of certain structures can only be determined by field strain measurements. One must bear in mind that a 10% variation in stress range leads to a 33% change in the number of permissible cycles. This is for a slope of -3 on the Wöhler diagram. For flatter slopes, the effect is even larger. It should be evident that it is very important to have as accurate a stress relationship as possible.

On the six structures considered, a special work train was used to establish a load to stress relationship at each gauge both statically and at speed.

This relationship was used to confirm an analytical theoretical model. Of the trusses examined, one behaved as a simple truss, another as a 3-dimensional space frame under moderate loading, and the others somewhere in between. The same was true of stringers in terms of continuity. None of this is surprising.

A large sample of regular traffic was then recorded and a relationship established between the measured weights and the strains at each gauge. The work train test and theoretical analysis were used as a check on this relationship.

The measured weights were then compared to the computerized data base. This data base is prepared by sales and operating personnel. The relationship, after accounting for dynamic effects, was quite good. This justified the use of the computerized data base to derive a load spectrum for the structure under consideration. The longest test recorded every train passing for a seven day period.

After the field test data is analyzed and tabulated, the load spectrum from the computer data base is applied to the relationship, linking it directly to the strain at each gauge location from the sample of regular traffic. In this way, average impact and sequence are automatically included. The work train tests are used solely as back-up and justification for various effects in the relationship.

At this point, either Miner's rule or a Root-Mean-Square stress range can be used with known Wöhler curves to predict the onset of fatigue damage.

The first two applications of this technique were able to show that several previously condemned spans of a bridge could be retrofitted in lieu of complete replacement. The savings to the Company was well over $50 million.

In another case, the measurement of lateral impacts was sufficient to explain several failures that occurred on structures less than 5 years old. Needless to say, details on other structures would be designed to avoid recurrence of the problem. The data gathered by the writer has been instrumental in preparing the new A.R.E.A. fatigue clauses and has led, together with the work of others, to a new awareness of the importance of good detailing and enlightened assessment of new and older bridge structures. Because of the fact that most older metal spans are made of material having very low fracture toughness, fatigue cracking can lead to brittle fracture. CN's fracture control plan is designed to avoid any serious collapses.

For many years, cracking of riveted structures was controlled by the inherent crack stoppers between the component parts of built up members (Figure 3) [7]. Inspection intervals were frequent enough to report any cracks, and corrective measures were taken before an adjacent part could develop its own cracks. The historic exception has always been rolled sections which generally did not have serious stress raisers combined with frequent high tensile stress excursions.

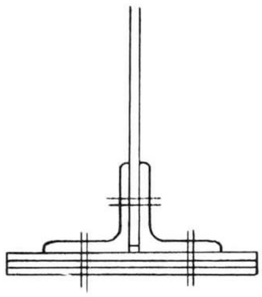

Fig. 3 Built up Member

It seems as though fatigue and fracture are a far more serious problem in welded structures [8] than in riveted structures. Part of this is due to the long period of experience with riveted structures during which most of the really bad details were eliminated, and part is due to the inherent component redundancy and somewhat lower rigidity of riveted structures. Unfortunately, welded structures tend to be less forgiving of small defects than riveted structures because they normally contain less excess material and because the welds themselves are points of rigidity and residual stress. Details which had been derived over many years of experience as adequate for riveted structures proved to be inadequate for welded structures.

With the advent of welded construction and its lack of component cracks stopping planes and its somewhat less tolerance for secondary out-of-plane displacements, the number of serious cracks requiring immediate attention has grown. Fortunately, with the advent of welding, CN Rail also went to very tough steels (Charpy values of 20 Joules at -17°C), with their large permissible critical crack sizes, and generally also went to more redundant (multi-beam) spans.

The first group of problems to turn up on our railway on welded structures were cracks at the bottom of stiffeners on skewed structures. Figure 4 is typical of this detail with a diaphragm or brace frame attached to the stiffener. Note that the stiffeners are not extended to the bottom flange. This was in blind obedience to the dictum of an early researcher in welded construction that one should not weld to the tension flange. Because of the stresses introduced by the differential deflections of the connected girders, and to small out-of-plane movements, the first cracks appeared in less than 5 years on heavily travelled lines.

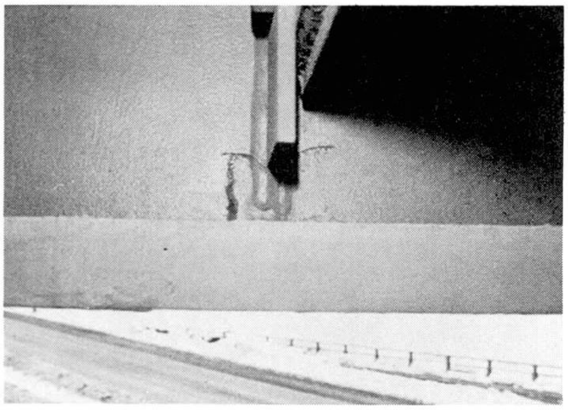

Fig. 4 "U" shape crack

The crack would start at the bottom of the stiffener and form a "U" shape crack around the bottom of the stiffener. If the original stiffener-web weld was of good quality, this crack would turn out into the web and slow down considerably.

After cracks have occured as described above, the next point of rigidity is between the web and flange. If there is any out-of-the plane of the girder motion, it is only a matter of time before cracks will occur in the web to flange weld below the stiffener (Figure 5). Stopping these cracks is very important.

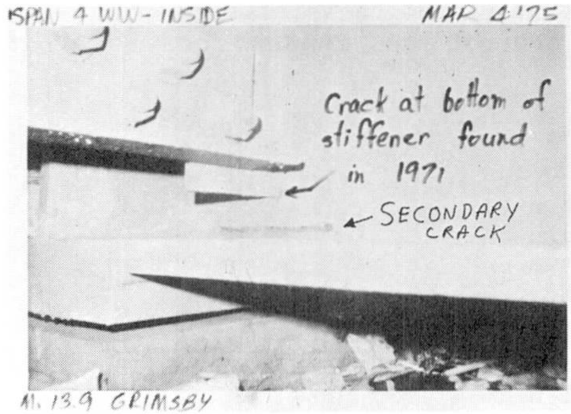

Fig. 5 Secondary Cracking

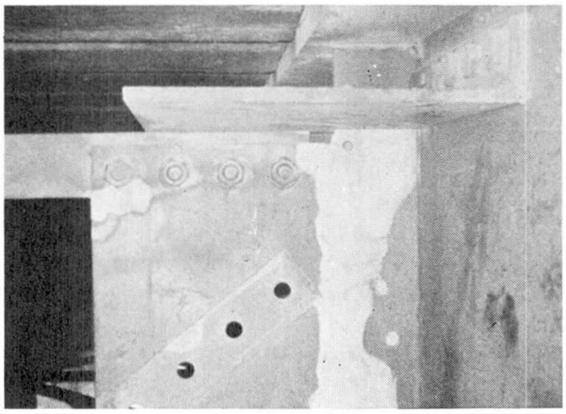

Fig. 6 Gusset to Stiffener

Similar cracks have occured on a few non-skew structures at the bottom of a stiffener simply because the stress range was too high. These take about 10 years to develope a sufficient number of load cycles.

Another detail that has given trouble is the connection of brace frame angles to stiffeners (Figure 6) with the gussets groove welded to the stiffener. Referring to Figure 7, the critical detail has a zero radius. The stress causing crack growth is the lateral force from trains. Since maximum lateral impacts often occur due to hunting (side to side snaking) of empty cars, it took less than five years for these cracks to develop.

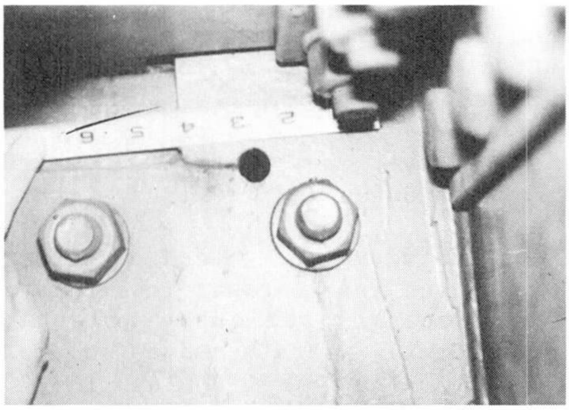

Fig. 7 Zero Radius

Longitudinal stiffeners are usually supplied in varying lengths and butt welded. Since they are usually in a compression area not much concern has been given to inspection in the past. At least one railway and several highway departments have suffered failures from defects in these welds. Although it is true that critical cracks exist only in tension areas; remember that most welds are tension areas because of residual stresses induced by the welding process. A crack can run along a weld until it reaches a tension area or until there is no material left [8].

2. REPAIRS

The subject of steel bridge repairs is just as important as initial design as often the cure is worse than the original problem [3]. The major problem with welded repairs to a riveted girder is that the weld destroys the initial component redundancy of the girder.

In the late 60's, it was decided to repair the corrosion that occurs in deck plate girders at the web-bottom flange junction on a number of our structures. Because of the production operation, patch plates were of various lengths, and were butt welded as required. After 10 years these welds started to crack under very low stress ranges. The maximum measured stress range was 36.2 MPa (5.25 ksi) with a mean peak per train of 31.92 MPa (4.63 ksi). There was little if any apparent dynamic augment.

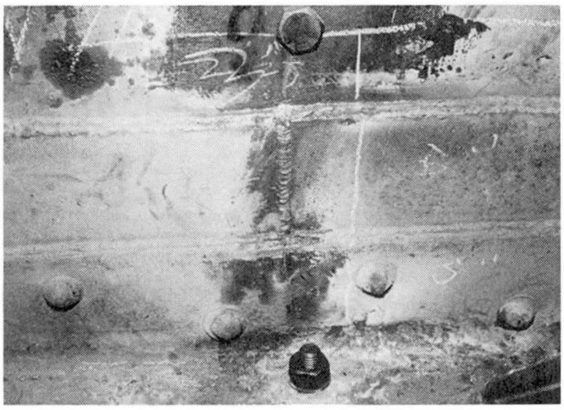

Fig. 8 Crack in Patch Plate

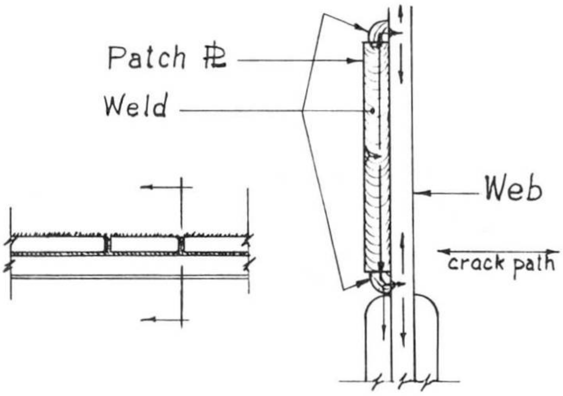

Fig. 9 Crack Growth

The cracks (Figure 8) in these welds were very hard to detect and were not detected until they had propagated from the patch plate welds into the web.

The crack path was as follows:

1. Crack initiation at a flaw.
2. Crack growth (Figure 9) in the weld upward and downward in those welds that did not have full penetration and horizontally into the web for those that did.
3. Growth from the level of the horizontal fillet welds into the web and bottom flange leaving multiple cracks.

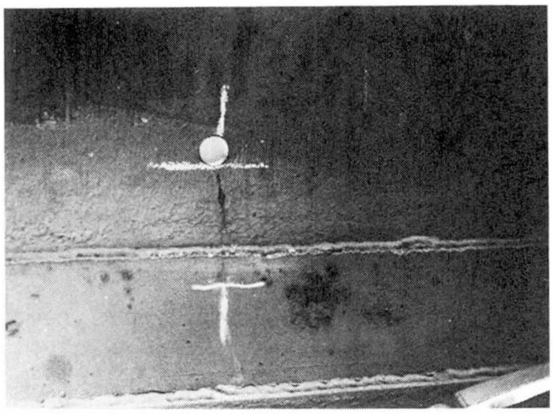

Fig. 10 Cracked Through

Figure 10 shows the same location looking from the inside of the girder. Note that there is no butt weld on the inside at the location of this crack. The crack has propagated from the opposite side.

Four crack fronts have been illustrated from the same initial crack. Depending on the degree of penetration, more crack fronts could theoretically develop. Hence, the repair procedure cannot consist solely in repairing the visible cracks as there may be other hidden cracks.

In the aircraft industry crack stoppers are used. This may consist of a line of closely spaced rivets, a stiffener or a band of much tougher material. In riveted or bolted bridge structures the interfaces between component parts act as crack stoppers. Join these with a weld and the crack stopper is by-passed. If this could lead to a catastrophic failure it must not be permitted.

3. STOPPING THE CRACK

The classic way of stopping a crack is to drill a round hole [9].

In some cases such as where a crack penetrates the stiffener to web weld a minimum of three holes must be drilled. The following procedure has been used successfully on at least two bridges:

1. Drill 7/16 diameter holes on both sides at the stiffener through the web at an approximate angle of 30° away from the stiffener (Figure 11) making sure

to get a truly circular hole with as few rough edges as possible.
2. The resulting hole in the web should be reamed from the outside using a reamer of the same diameter throughout.
3. Attention should be taken so that all rough edges are made smooth even though a truly circular hole is not obtained. Hand filing may be necessary.

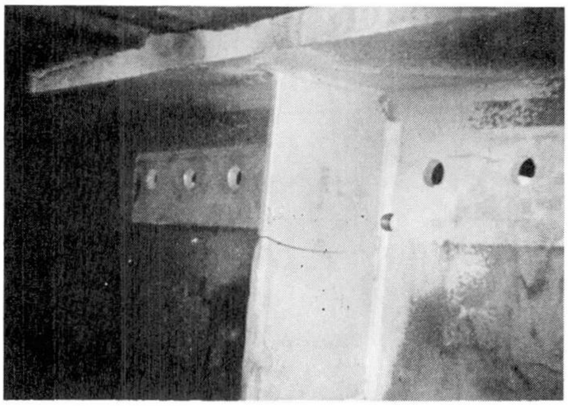

Fig. 11 Hole at Stiffener

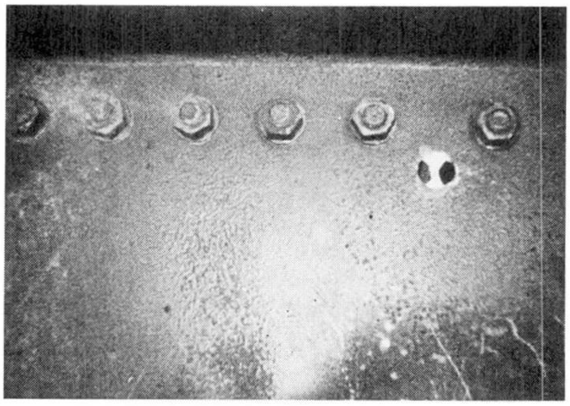

Fig. 12 Other Side

Figure 12 was taken after the two holes drilled from the inside had been reamed to one hole from the outside flush with the stiffener on the inside. The white metal along the vertical centerline of the hole is the stiffener.

In cases where the round hole remains an unacceptable fatigue detail, and where it is physically possible, a high strength bolt can be placed ensuring that the non-burr side of the washer is placed against the steel. The clamping force of the bolt will ensure a better detail and will prevent any further crack propagation should the crack tip have penetrated the far side of the hole. After stopping the crack, attention can then be focused on the repair.

4. RETROFITS

Unfortunately in many cases the only adequate cure is not to have the detail in the first place. If the crack isn't going to go anywhere, and if it has not damaged the structure too severly, it is best left alone as the retrofit may make things worse. Cracks at the bottom of stiffeners tend to be in this category.

Repairing the cracks that occur between the flange and web is extremely important. The general procedure for any crack is to:

1. Use a chisel to vee out the crack,
2. Fill the resulting groove using a proper electrode, and
3. Grind flush.

If the structure cracked because it was restrained from movement and if movement is essential to its function, as in the case of a skew-girder, then a repair of the existing detail will be of no value. In fact it may not crack in the same place next time but at a more serious location. In addition to the obvious differences in vertical deflection of skewed girders, one should consider conceptually that torsion in a girder will occur in the part most capable of rotating. The flange is generally much stiffer than the web, particularly in the space between the stiffener and the bottom flange, and any girder rotation (out-of-plane movement) will be forced to occur in this small space. Needless to say the stresses will be enormous. To solve the problem, the stiffeners must be run down to the bottom flange, or cut back far enough to relieve the stresses, or the source of rotation removed. If the stiffeners are connected to the bottom flange in a high positive moment area, this may entail a reduction in capacity.

At this stage an economic evaluation must be made as to the desirability of altering the detail that causes the crack, or of being prepared to accept that similar cracks will re-occur or worse still that eventually the structure may have to be replaced. For CN's rates of return, it is more ecomomical to leave the detail alone if the repair will last at least 10 years.

Future technology, in particular gas-tungsten arc Remelt [10] may make future repairs easier and more reliable.

In the case of the gussets groove welded to the stiffener, since these developed in less than 5 years, they could be expected to re-occur in the same time frame if repaired. It was decided to replace the butt welded detail with a bolted detail for the brace frames. Prior to doing this, all cracks were repaired by welding and the top and bottom three inches of the stiffener where the gusset place was removed were ground smooth to get rid of any incipient cracks.

In the case of longitudinal stiffeners, if the crack is arrested before it runs, it is usually left alone.

When patching girders, it is a simple matter to run patch plates far enough so that the girder stress will be small enough to permit such a detail.

Whenever a sharp notch is noticed in a member, if it is in a highly stressed area and if it is not on a nearly abandoned branch line, the notch is ground out at the first opportunity. The same should be done for tack welds which are serious stress raisers.

5. REDUNDANCY

Most riveted structures were internally member redundant in that they were fabricated of several components (Figure 3). Their advantage is that cracks do not jump from piece to piece. The chord member of the truss shown in Figure 13 cracked on one side, but not on the other side of the member, and carried rail traffic for some time before detection and repair. In a welded box like member the crack would have propagated all around.

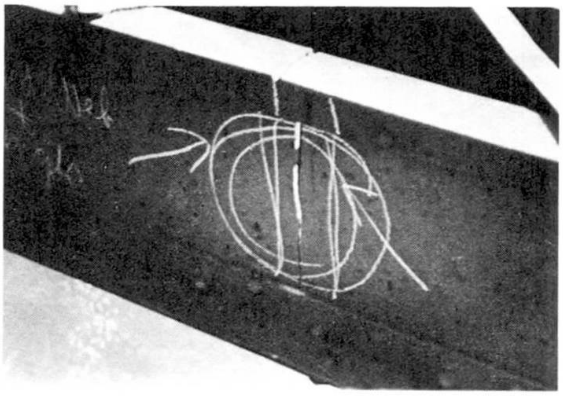

Fig. 13 Crack in chord

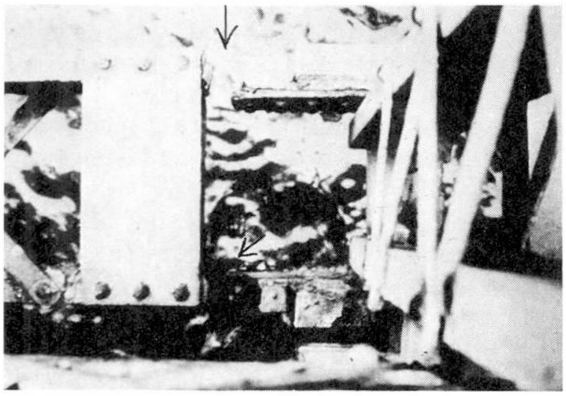

Fig. 14 Chord Severed

Both riveted and welded structures can be redundant by having multiple member load paths as in a multi-beam bridge. Bracing systems can add to redundancy.

Most trusses are multi-load path structures. The truss in Figure 14 had its bottom chord completely severed and yet remained standing because of the alternate load paths provided by the bracing, floor system, etc. The truss shown in Figure 15, which did not have adequate bracing, did not remain standing when one of its diagonals was severed by a shifting load. It is unfortunate that it didn't have sufficient redundancy.

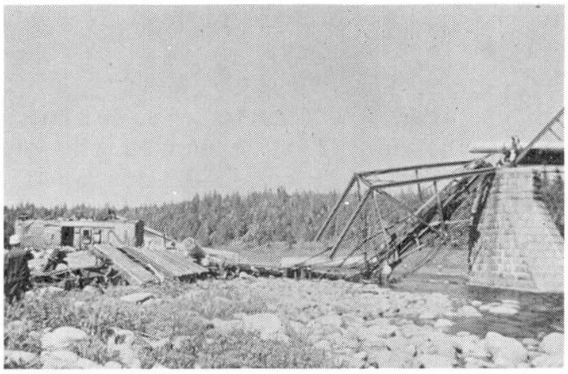

Fig. 15 Insufficient Redundancy

On the other hand, if one considers the deck plate girder bridge, which on railways is generally a two girder system, there is very little member redundancy and virtually no component redundancy in a welded structure. In this type of structure if a crack starts due to an accidental impact, or to a nick fabricated in or due to some less than ideal detail, it may run until the structure fractures [8]. In most cases the bracing will be hard-pressed to carry the dead load, let alone anything like full live load for any length of time. Inspection intervals must be frequent enough to spot these fractures otherwise catastrophic failure will result.

In the typical riveted structure, a crack, from whatever detail, will propagate only within the component that had cracked. For example, in a typical plate girder if a crack starts in a flange angle it will not transfer to the cover plates, web or the opposite flange angle. Another crack may develop, but this will take time and further application of load. Inspection should be able to detect the initial crack and initiate necessary repairs before a serious problem develops. This type of component redundancy permits much greater inspection intervals and permits more time to schedule repairs. It often permits the deferral of repairs until other items combine to make it worthwhile to send in a repair crew.

In order to emphasize the previous discussions, it is instructive to illustrate a few failures where redundancy has saved welded structures or where component redundancy has saved riveted structures.

A rather striking example (Figure 16) is this multi-beam welded structure with a composite deck which was hit by the top of a backhoe. Although several girders were badly damaged, the structure did not collapse under train traffic. It is known that several trains crossed on the adjacent track after the accident. Imagine what could have happened if that had been a two girder non-composite welded system.

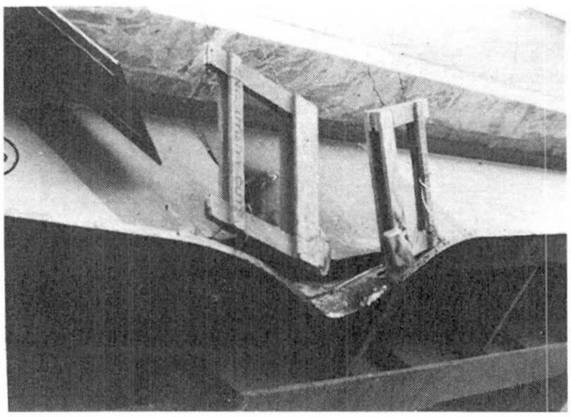

Fig. 16 Hit by Vehicle

The skewed multi-beam composite structure shown in Figure 5 cracked due to a fatigue related failure. Throughout the repairs, which were delayed for over a year, regular train traffic was permitted. This could not have been permitted on a non-redundant load path system.

This (Figure 17) structure cracked due to torsional loading caused by constant train breaking. Although the floor beams are not welded there is no component redundancy as the beams are rolled. Nevertheless, the redundancy of the deck system has permitted regular train operations for over a year with 4 adjacent floor beams failed.

The type of repair shown in Figure 18, the addition of welded plates to an eye bar member could be a disaster in a non-redundant load path structure. The crack is illustrated by the light line of magnaflux at the toe of the weld. In this bridge, because of its multiple load paths, collapse will not occur if one of these members cracks through.

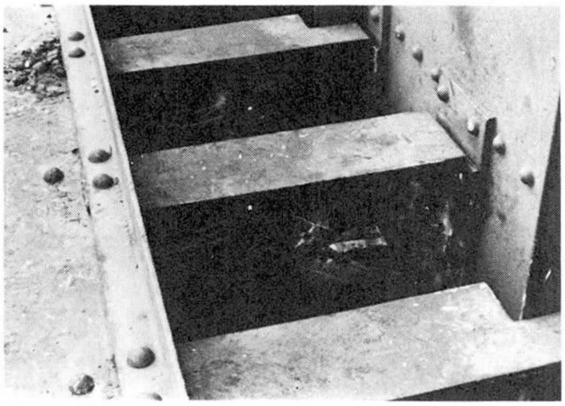

Fig. 17 Torsional loading

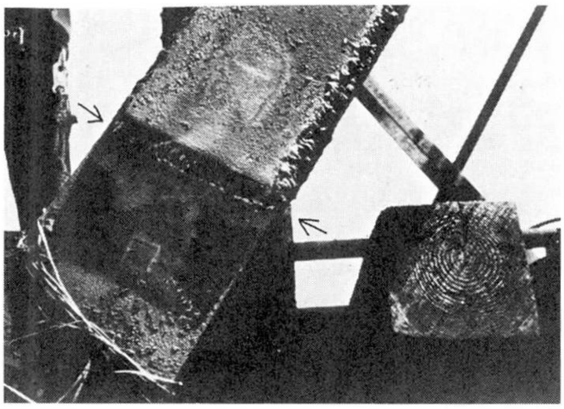

Fig. 18 Eye Bar Weld

With the aid of the new fatigue design rules [5], and a much better understanding of out-of-plane displacements [11], load spectrums, etc., many of our past problems can be covered by design. It must be emphasized that adequate design, particularly for non-redundant load path welded structures means good design, fabrication and inspection.

Our current bridge fracture control plan consists of periodic inspection for component redundant or member redundant spans as these spans may be considered fail safe due to their alternate load paths, and inspection plus live load restriction where necessary on non-redundant load path spans which is a safe life approach.

In the first case we let the structure crack and then take action as the spans are fail safe, in the second case, loads or sufficient cycles of loading that could cause initial cracks to form are not permitted, thus ensuring safe life.

6. FIRES

In a country as vast as ours bridge fires are a serious problem. The problems of getting a structure back into service after a fire are staggering [12]. Locked in residual stresses in apparently simple structures due to the limitations to expansion because of abutments, and other members such as bracing systems can and do approach the yield point. As a case in point the chord of one of CN's trusses cracked (without live load) on cooling after a fire and continued to crack due to daily thermal cycles for the next three weeks. Since many of the railway's structures are supported by open timber cross ties, the mechanical damage when these burn and a loaded car drops down can completely ruin a span.

On a simple three pinned arch [12], the measured residual stresses in the top chord after a fire reached the yield point. Again the internal redundancy preventing large movements was the cause.

As part of our fire prevention control plan, a policy was developed to eliminate major timber structures on main lines. Subsequently, a program of coating timber decks was developed using a propriety fire retardent material.

On those structures which can support the weight, prestressed concrete deck slabs are used to replace the timber decks.

Recently on long structures, the expedient of inserting fire breaks has been considered. On some of our structures in the Rockies, the cross-ties are 0.36 m x 0.56 m (14" x 22") supported by trusses 3.96 meters (13 feet) apart with no intervening floor system. The replacement in kind of this type of deck is costly. With this in mind and in view of the tremendous amount of potential structures, an inexpensive fire break which would assist in the majority of cases was required.

On one region, the expedient of placing an asbestos barrier between spans extending below the rail for a distance of 1.5 meters (5 feet) has been tried. Installation costs are roughly $100 and although the level of protection is not high, it is better than nothing.

Although these measures will not prevent fire damage, it is hoped that they will contain some of it.

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