19. J. D. Pault. Composite Action in Glulam Timber Bridge Systems. Colorado State Univ., Ft. Collins, MS thesis, 1977.
20. D. B. Van Dyer. Strength of Built-Up Timber Columns. Nova Scotia Technical College, Halifax, PhD thesis, 1976.

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Some Examples of Detection and Repair of Fatigue Damage in Railway Bridge Members

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Examples of details that have caused fatigue damage in recently designed railway structures are given. Procedures for arresting crack growth and some repair details are described. Emphasis is placed on damage caused by secondary and out-of-plane effects often not considered by designers.

This paper is concerned with some details that have caused fatigue damage in recently designed railway structures and that may not be generally recognized as inadequate by the design profession.

Sufficient experience had been gained by World War I to determine the adequacy of various riveted details. In general, any fatigue problems on riveted structures experienced on our railroad are caused by extremely high cycle fatigue, overloading, or details that are well known to be inadequate. Detailing of riveted railway bridge construction has not changed appreciably during the last 40 years. This, together with the greater concern for the teaching of overall analysis brought about by the digital computer, has led to a generation of engineers unconcerned with details. In many cases, major details are left to the discretion of the draftsperson.

Unfortunately, welded structures tend to be less forgiving of small defects than are riveted structures because they normally contain less excess material and because the welds themselves are points of rigidity and residual stress. Details that had been proved over many years of experience to be adequate for riveted structures are proving to be inadequate for welded structures. In particular, much greater attention must be paid to out-of-plane stresses and secondary effects.

It is only within the last few years that, as a result of some failures (1), new research and the study of fracture mechanics (2, 3) have permitted including in the codes more detailed advice on the suitability and limitations of various welded details.

On our railway, the change from riveted to welded designs took place in the early 1960s. The first designs incorporated details patterned after existing riveted construction.

DETAILS

1. As an extreme example and to make a point, consider the multibeam structure shown in Figure 1. The cover plates are attached by intermittent welds. Each small length of weld is designed to replace a certain number of rivets. The current codes classify this as an E-detail because of the lack of adequate test data. Nevertheless, in this case, it is probable that higher strength exists because the intermittent welds are continuing so that the connected plates are about equally strained and because the welds are well made. This last point is crucial in evaluating details. Despite the fact that the structure has been in service since 1961, because of its redundancy, the actual average root-mean-square stress range is far below the limits of category E. Unfortunately, defective details take time to become evident and, because of redundancy, may never show up.

2. On our railway, the first group of problems to develop on welded structures were cracks at the bottoms of stiffeners on skewed structures. Figure 2 shows a typical example—a diaphragm or brace frame attached to a stiffener in which the stiffeners are not extended to the bottom flange. This was in blind obedience to the dictum of an early worker 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 by small out-of-plane movements, cracks appeared in less than 5 years on heavily traveled lines. This type of crack begins at the bottom of the stiffener and then forms a "U" shape around it. If the original stiffener-web weld is of good quality, the crack turns out into the web and slows considerably. A temporary cure is to drill a round hole containing the crack tip. However, if the web-stiffener weld has a series of surface toe discontinuities (undercut or lack of penetration), the crack can run up the web, which causes the girder to split in half. Fortunately, our railway has not experienced such a failure. Figure 3 shows a crack following a weld upward. A circular hole was drilled to prevent any further propagation.

3. After cracks have occurred as described above, the next point of rigidity is that between the web and flange. If there is any motion out of the plane of the girder, it is only a matter of time before there will be cracks in the web-to-flange weld below the stiffener (Figure 4). Stopping these cracks is very important. Figure 5 summarizes the problems with this type of cracking.

4. Similar cracks have also occurred on a few non-skewed structures at the C-detail at the bottom of the stiffener simply because the stress range was too high.

These required about 10 years to develop a sufficient number of load cycles.

5. Another detail that has given unexpected trouble is the connection of brace frame angles to stiffeners (see Figure 6) that have the gusset groove welded to the stiffener. As shown in Figure 7, the critical detail has a zero radius (category E). The stress that causes crack growth is the lateral force from trains. Because the hunting (side-to-side snaking) of empty cars causes frequent maximum lateral impacts, it took less than 5

Figure 1. Multibeam structure that has cover plates attached by intermittant welds.

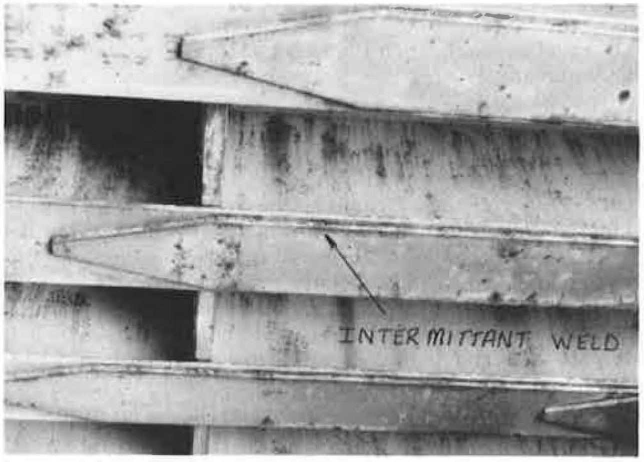


Figure 2. Crack at bottom of stiffeners on skewed structure.

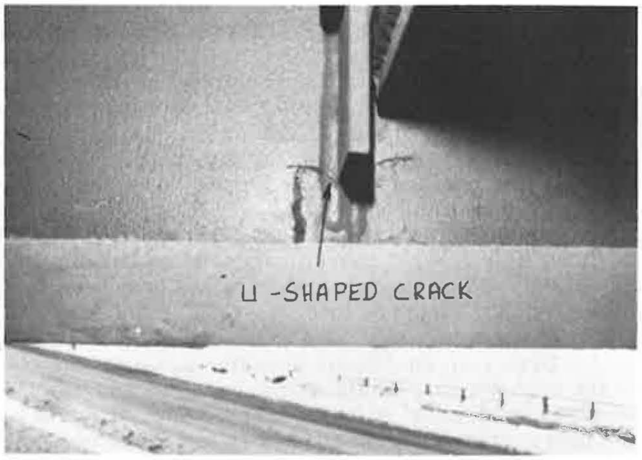


Figure 3. Crack following weld upward.

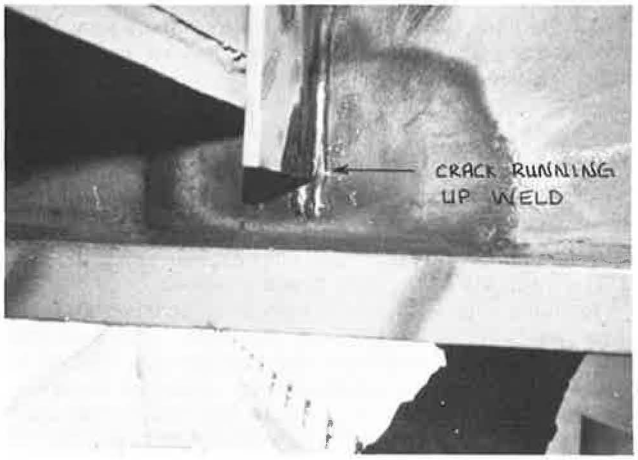


Figure 4. Crack in web-to-flange weld below stiffener.

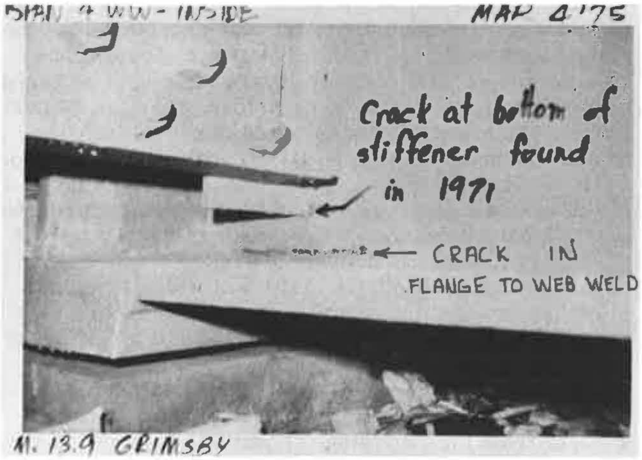


Figure 5. Summary of web-stiffener-flange cracking.

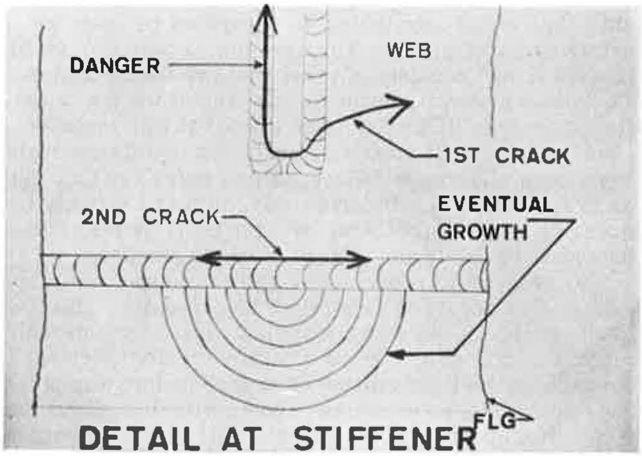
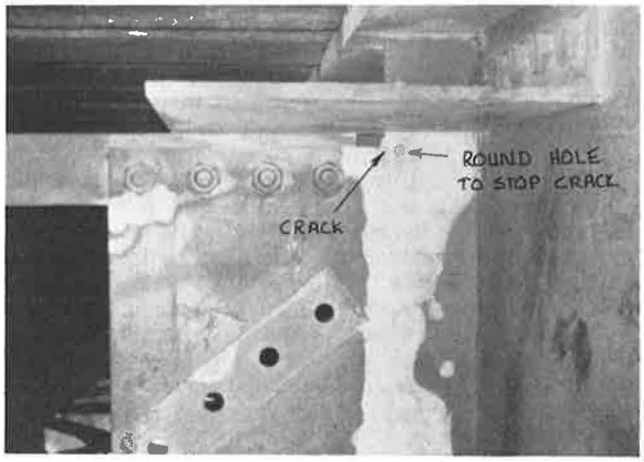


Figure 6. Connection of brace frame angle to stiffener that has gusset groove welded to stiffener.



years for these cracks to develop. The time from the discovery of the first crack until the development of more than 400 on the same structure was less than 1 year. The danger is that, when the crack reaches the stiffener-to-web weld, if there are a few microdiscontinuities in that weld, the crack can run down the weld and split the web in half.

6. Longitudinal stiffeners are usually supplied in varying lengths and butt welded. Because they are usually in a compression area, not much concern has been given to inspection of them in the past. At least one railway and several highway departments have suffered failures from defects in these welds. Thus far, failures in this type of detail have initiated in overall tension areas. Although it is true that critical cracks occur only in tension areas, most welds are tension areas because of the 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. A crack (see Figure 8) can grow slowly in the butt weld, propagate along the longitudinal-stiffener-to-web weld, and then run down a vertical-stiffener-to-web weld and split the girder in half (4).

7. Intersecting welds pose serious problems. Fortunately, these have generally been prohibited on our railway from the beginning. There is, however, a very interesting case in the literature (5).

8. Patch plates and similar repairs (see Figure 9) always have an E-detail at the corners. There have not been significant failures on riveted structures at patches because (a) corrosion tends to erode the crack tip and slow the rate of growth and (b) redundant components act as crack stoppers. For example, a typical riveted tension flange consists of a web and two flange angles. Corrosion generally occurs in the web at the toe of the flange angles. If a patch plate placed at this location causes a crack, it must also crack the two flange angles to cause a significant failure. Rivet holes can also act as crack stoppers. Unfortunately, this is not likely to occur on welded structures because there is less component redundancy and almost no crack stoppers.

9. Figure 10 illustrates a rather unexpected fatigue failure that occurred as a result of corrosion. The I-beam stringers shown were subjected to a considerable amount of oil, grit, and such because trains often sat for quite some time waiting clearance and also engaged in frequent stops and starts during switching operations. The bottom portion of the web above the bearing corroded sufficiently so that the out-of-plane bending stresses reached a critical level. Lateral forces caused a crack to run along the fillet between the flange and web. Within a very short time, the bottom flange, which suddenly had to act as a beam, tore through. If bearing stiffeners had been placed as required by the code, the small lateral movements could not have occurred and the web could literally have rusted to nil before failure. The time to failure of these stocky web sections was about 15 years.

10. In many beam-and-deck-plate girder spans that have open timber decks, the inside part of the top flange must resist the bending of the ties (Figure 11). A larger than average tie will carry excess load. If there are insufficient stiffeners or if the stiffeners do not fit snugly to the flange, the flange will crack. If the crack is not arrested, it can turn down at the first vertical weld and split the web in half.

11. At stringer or floor beam connections, inadequate copes can approach an E-detail. Under normal traffic, these will not behave as pin connections but as nearly fixed. The resulting tension can cause a crack to propagate (see Figure 12). The beam is resting upside down.

12. Welds and notches that do not meet the design codes are a source of almost certain future cracking, as are notches caused by impacts from trucks (see Figure 13), loose shipments (see Figure 14), or even ships. The rapid rate of dynamic loading caused by a shipment collision usually causes immediate brittle fracture. An example of the damage that a loose shipment can cause is shown in Figure 15; the bridge truss collapsed when one of the tension members was severed by a load of culvert pipe.

On rarely used branch lines, restrictions can be relaxed considerably.

STOPPING CRACKS

The classic way to stop a crack is to drill a round hole (6). This converts a running crack that must be considerably worse than an E-detail to a B-detail if the hole is carefully reamed or at least to a D-detail (unless the workmanship is really sloppy). On our railway, a 1.27- to 22.7-cm (0.50- to 0.875-in) diameter hole is specified, depending on how easy it is to get at the crack tip. It is imperative that the hole contain the crack tip. If there is any doubt, it is better to lead the crack. Figure 16 shows a hole that did not contain the tip. The crack propagated from the hole edge where it was not visible when the hole was drilled to that shown in less than 4 weeks.

On flat surfaces, a Halec eddy-current crack detector is used to find the crack tip because this avoids removing the paint. On one bridge, this saved the cost of the machine. Note that the crack tip may not have broken the surface paint.

In other cases, the paint is removed and either a magnetic particle test or a good visual examination with is a magnifying glass are used. Dye penetrants do not offer a significant benefit over a magnifying glass in this type of work and, because of generally windy conditions, are messy to work with.

In some cases, such as cracks that penetrate 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 1.11-cm (7/16-in) diameter holes on both sides at the stiffener through the web at an approximate angle of 30° away from the stiffener (Figure 17a), ensuring that the hole is truly circular and has as few rough edges as possible.

2. Ream the resulting hole in the web from the outside by using a reamer of the same diameter throughout until the hole reaches the stiffener and the surfaces are left smooth.

3. Be sure that all rough edges are made smooth even if a truly circular hole is not obtained and if hand filing is necessary.

Figure 17b shows the two holes drilled from the inside as shown in Figure 17a before the web was reamed to produce one smooth hole. Figure 17c shows the appearance 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 is an unacceptable fatigue detail and it is physically possible, a high-strength bolt can be placed to ensure that the nonburr side of the washer is placed against the steel. This will ensure a B- detail or better and prevent further crack propagation because of the clamping force, should the crack tip have

Figure 7. Critical detail from Figure 6.

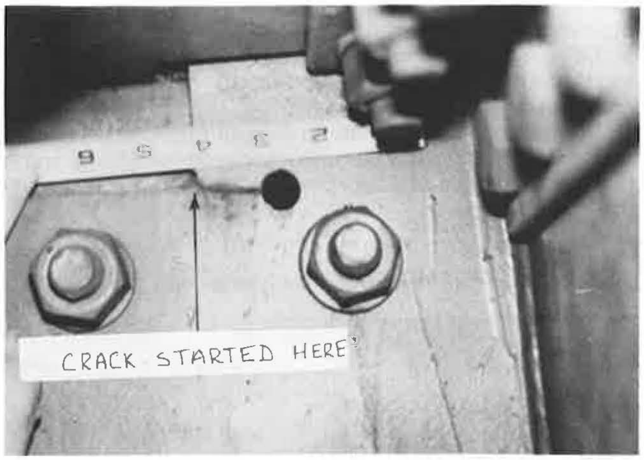


Figure 8. Crack growth from butt weld.

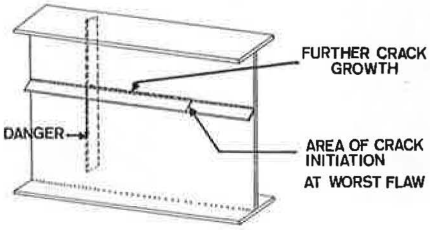


Figure 9. Typical repair section.

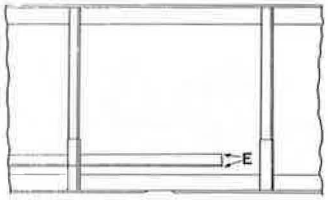


Figure 10. Corrosion-caused crack.

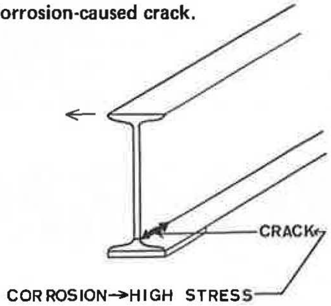


Figure 11. Crack caused by insufficient or badly fitting stiffeners.

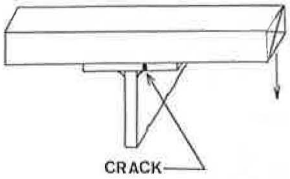


Figure 12. Crack caused by inadequate cope.

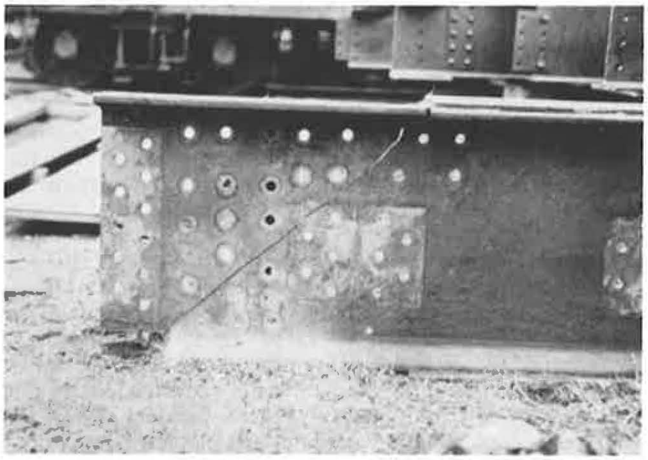
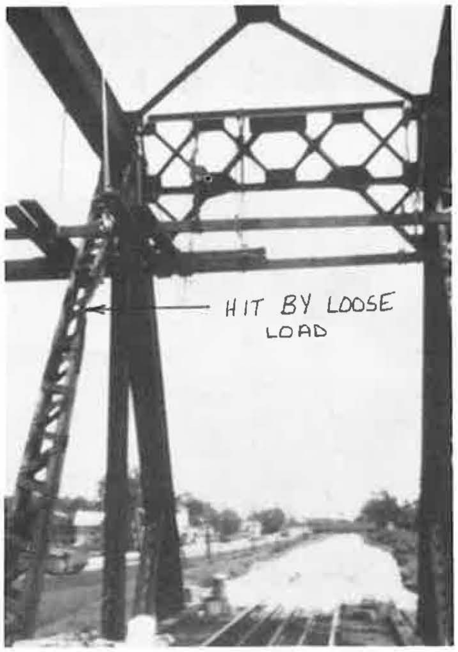


Figure 13. Notch caused by impact from truck.

Figure 14. Notch caused by impact from loose shipment.



penetrated the far side of the hole. After stopping the crack, attention can then be focused on the repair.

REPAIRS

Unfortunately in many cases, the only adequate cure is not to use the detail in the first place. If the crack is not going to propagate and has not damaged the structure too severely, it is best left alone because the repair may make things worse. Cracks at the bottoms of stiffeners tend to be in this category.

If the structure cracked because it was restrained from movement and movement is essential to its function, as in the skewed-girder case, then a repair of the existing detail will be of no value. In fact, the next crack may be not in the same place but at a more serious location. A quick calculation of the stresses in a squared brace frame between skewed girders shows that these stresses will be very high and cannot be taken by the web alone (Figure 18). In addition to the obvious differences in vertical deflection of skewed girders, one should consider conceptually that the 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 (Figure 4). Needless to say, the stresses will be enormous. To solve the problem, either the stiffeners must be run down to the bottom flange, or they must be cut back far enough to relieve the stresses, or the source of rotation must be removed. If the stiffeners are connected to the bottom flange in an area that has a high positive moment, this may entail a reduction in capacity. In some cases of composite deck slabs, a repair is not worth doing. The cracked detail will suffice for the one-shot accidental load.

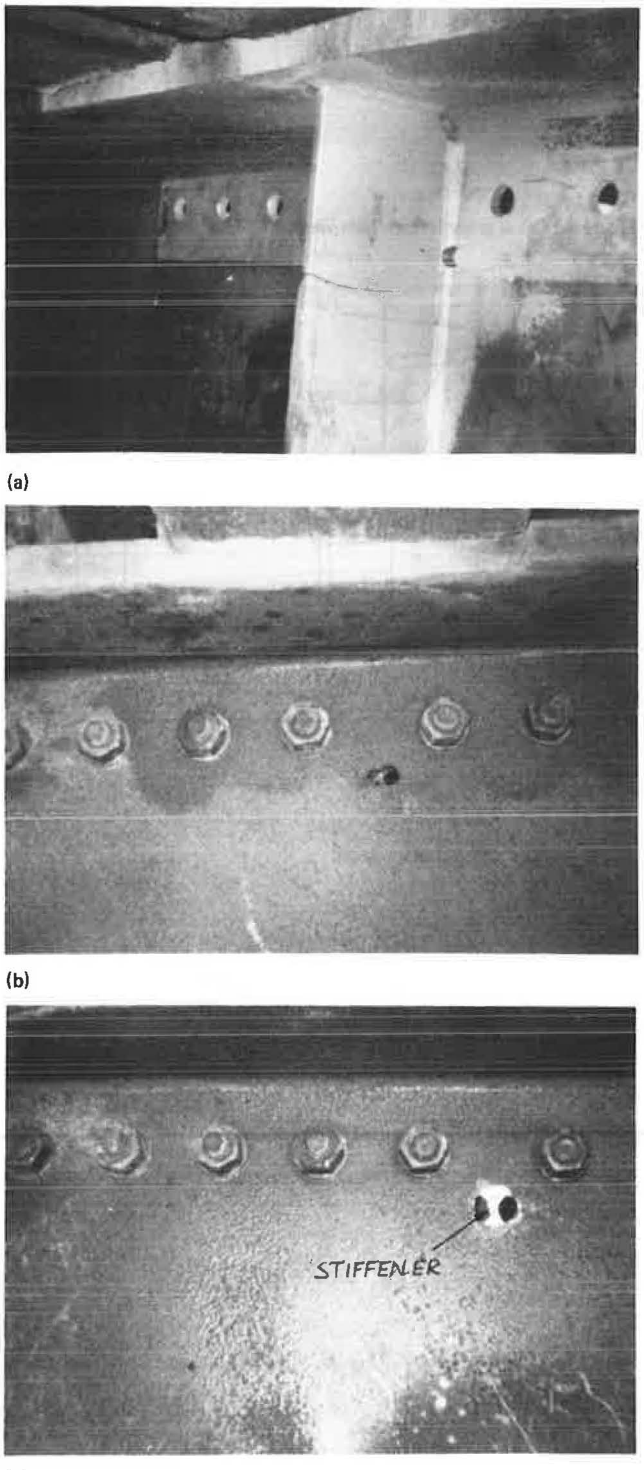
Figure 15. Truss collapse caused by impact from loose load.

Figure 16. Hole that did not contain crack tip.

Similar problems occur in curved girders.

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

Figure 17. Procedure for stopping crack propagation.



(a)

(b)

(c)

Figure 18. Stresses in squared brace frame between skewed girders.

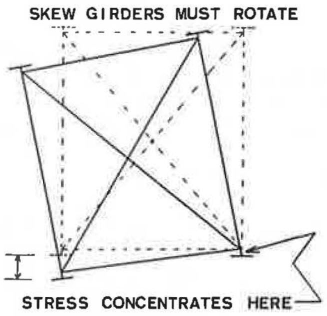


Figure 19. Procedure for repairing crack in gusset groove welded to stiffener.

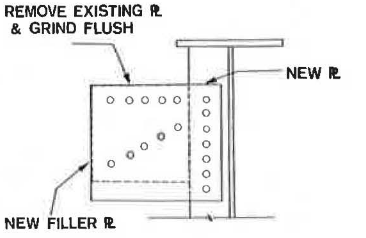
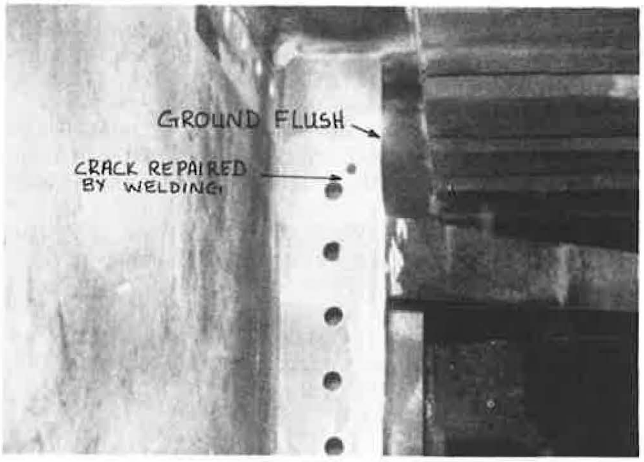


Figure 20. Illustration of procedure shown in Figure 19.



1. Use a chisel to vee out the crack,
2. Fill the resulting groove by using 7018 electrodes, and
3. Grind flush.

At this stage, an economic evaluation must be made as to the desirability of altering the detail that caused the crack, or of being prepared to accept that similar cracks will reoccur or, worse still, of accepting that eventually the structure may have to be replaced. In any organization where capital is in short supply, demanding high rates of return (15 percent or more), the usual decision is to leave the detail as is.

For the railway's rates of return, it is more economical to leave the detail alone if the repair will last at least 10 years.

New technologies, in particular the gas-tungsten arc remelt (7), may make future repairs easier and more reliable.

The detail of the gusset groove welded to the stiffener developed problems in less than 5 years; thus, problems with these details can be expected to reoccur in the same time frame if they are repaired. Therefore, it was decided to replace the butt-welded detail with a bolted detail for the brace frames (Figure 19). Before doing this, all cracks were repaired by welding (Figure 20) and the top and bottom 7.6 cm (3 in) of the stiffener, where the gusset was removed, were ground smooth to remove any incipient cracks.

Figure 21. Procedure for repair of cope that has small cracks.

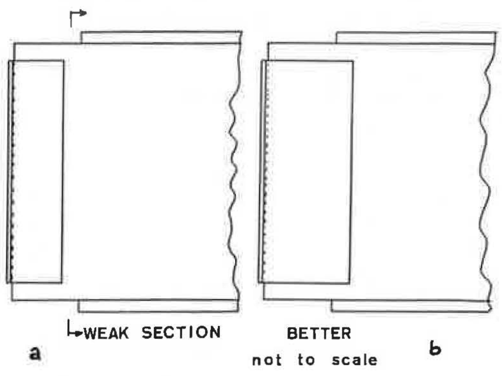


Figure 22. Sharp notch in member.

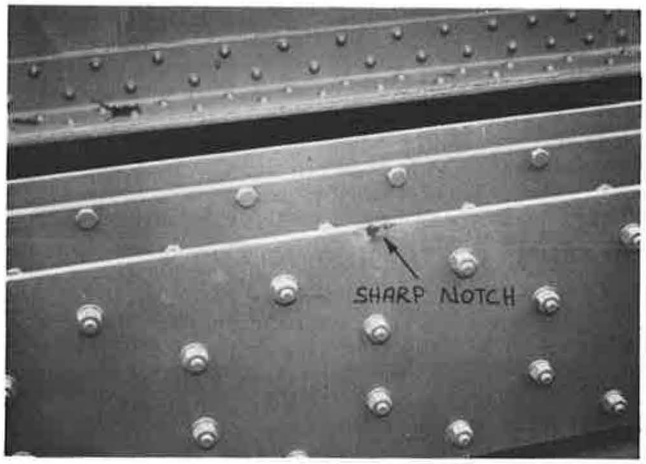
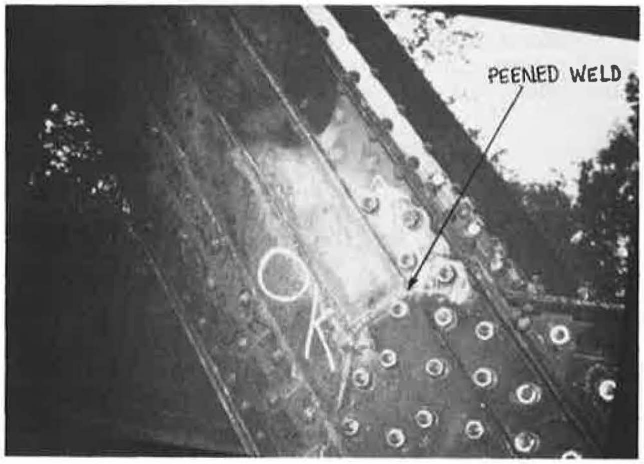


Figure 23. Truss repaired by welding and peening.



In the case of longitudinal stiffeners, if the crack is caught before it propagates, 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 an E-detail.

The corroded web was repaired by welding the flange to the web and adding a bearing stiffener. Those in which the flange had cracked through were replaced.

Failures in top flanges caused by transverse tie loading are rewelded. In some cases, stiffeners are added. These are welded to the top flange and not simply ground to fit because there is evidence (8) that fitting stiffeners is not adequate.

In the case of copes, those that have small cracks or none at all are ground to a smooth radius, which eliminates the problem. Others are handled in a variety of ways, taking care to use details that are adequate for the negative bending that designers do not calculate, but that actually occurs. The detail shown in Figure 21a is poor because there is a relatively highly stressed E-detail. A better solution is to use an oversized connection and not stiffen the joint in a prying sense (Figure 21b). This reduces the stress considerably at the notch.

Whenever a sharp notch is noticed in a member (Figure 22), if it is in a highly stressed area and if it is not on an almost abandoned branch line, it should be ground out at the first opportunity. The same should be done for tack welds, which seriously increase stress.

Several years ago, some strengthening was done to a truss (Figure 23). The welding was quite poor, and several cracks were detected on the surface. Some of the rough spots were removed and rewelded, and the entire weld was peened with a ball peen hammer. Sufficient peening was done to flatten any surface cracks and perhaps introduce a certain amount of residual surface compression. In theory, this should prevent surface crack growth. The structure has been in service since 1972 on a lightly traveled branch line in northern Vermont; it is too early to tell whether the desired effect was achieved. However, hand peening is extremely time consuming and unreliable if more than a short length of weld is involved.

For riveted connections, the expedient of replacing rivets that may be as bad as a D-detail by high-strength bolts (9), a B-detail, has been used after repairing the crack. As mentioned above, if the crack is still within the area of the washer, the residual compression induced by the bolt will prevent it from growing.

A note of caution is in order. There was one case in which a connection was badly underdesigned, and rough calculations that were inconclusive as to the number of cycles indicated that only a B-detail could have lasted so long. But after two rivets popped, it was quite clear that those rivets were functioning as high-strength bolts. Obviously, replacement with bolts would offer no advantage. The structure was replaced.

REFERENCES

1. J. W. Fisher and I. M. Viest. Fatigue Life of Bridge Beams Subjected to Controlled Truck Traffic. Proc., Seventh Congress, International Association of Bridge Structural Engineers, 1964.
2. J. W. Fisher, K. H. Fank, M. A. Hirt, and B. M. McNamee. Effect of Weldments on the Fatigue Strength of Steel Beams. NCHRP, Rept. 102, 1970.
3. J. W. Fisher, P. A. Albrecht, B. T. Yen, D. J. Klingerman, and B. M. McNamee. Fatigue Strength of Steel Beams With Transverse Stiffeners and Attachments. NCHRP, Rept. 147, 1974.
4. J. W. Fisher, M. D. Sullivan, and A. W. Pense. Improving Fatigue Strength and Repairing Fatigue Damage (Appendix F: Analysis of Crack Growth in the Quinnipiac River Bridge). Lehigh Univ., Bethlehem, PA; NCHRP, Final Rept. of Project 12-15(1), Dec. 1974.
5. J. W. Fisher, A. W. Pense, and R. Roberts. Evaluation of Fracture of Lafayette Street Bridge. Journal of the Structural Division, Proc., ASCE, Vol. 103, No. ST7, July 1977, pp. 1339-1358.
6. S. Timoshenko and J. N. Goodier. Theory of Elasticity. McGraw-Hill, New York, 2nd Ed., Chap. 4, 1951.
7. J. W. Fisher, A. W. Pense, and R. E. Slockbower. Retrofitting Fatigue-Damaged Cover-Plated Bridge Members. Paper presented at the 57th Annual Meeting, TRB, 1978.
8. D. A. Demo and J. W. Fisher. Analysis of Fatigue of Welded Crane Runway Girders. Journal of the Structural Division, Proc., ASCE, Vol. 102, No. ST5, May 1976, pp. 919-934.
9. H. S. Reemsnyder. Fatigue-Life Extension of Riveted Connections. Journal of the Structural Division, Proc., ASCE, Vol. 101, No. ST12, Dec. 1975, pp. 2591-2608.

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Stiffening the Manhattan Bridge

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The Manhattan Bridge is a suspension bridge located in New York City. Its lower level carries three lanes of traffic at the center of the bridge and two transit tracks on either side. Its upper level carries two lanes of traffic over each set of tracks. The loads are supported by four cables suspended over four stiffening trusses. After the bridge was opened to traffic in 1909, breaks appeared in the upper laterals. Substituted larger members also broke. The broken pieces were hazardous to the trains, which required the removal of the entire top lateral system. The breaks were attributed to torsional stresses induced by eccentric transit loads. Because it has no top laterals, the bridge has little torsional stiffness and large vertical and lateral motions occur between adjacent trusses during passage of trains, which causes many maintenance problems. Load tests for stresses and deflections were performed on the bridge, and a single-plane 50-scale model, 18 m (59 ft) long was constructed to duplicate the motions and stresses of the prototype. Schemes for stiffening the bridge—stays radiating from the tower tops to the stiffening trusses, tie cables that had small sags from anchorage to anchorage, diagonal ties between the cables and the stiffening truss, and side-span supports—were tested on the model. The side-span supports are efficient and economical in reducing deflections at the main-span center and almost eliminate deflections of the side spans. The tower stays and diagonal ties at the center of the main span are effective in reducing deflections of the main-span quarter points.

BACKGROUND

The Manhattan Bridge is the middle one of the three suspension bridges that connect lower Manhattan to