deterministic relations between the projected overload configurations and the type and amount of damage that the bridge superstructure will sustain. A parametric investigation into this for prestressed concrete I-beam bridges has been completed (42).

Similar developmental research for steel-girder bridges is under way. Parametric investigations of bridge superstructures subjected to predetermined overload configurations will permit establishment of a sufficient data base that relates load configurations to the type of damage that the superstructure will exhibit. This, in due course, can and will provide the necessary information for the establishment of the load levels.

39. C. N. Kostem. Overloading Behavior of Beam-Slab Type Highway Bridges. Fritz Engineering Laboratory, Lehigh Univ., Bethlehem, PA, Rept. 378B.8, July 1977.
40. W. S. Peterson and C. N. Kostem. The Inelastic Analysis of Slab Highway Bridge Superstructures. Fritz Engineering Laboratory, Lehigh Univ., Bethlehem, PA, Rept. 378B.5, March 1975.
41. W. S. Peterson and C. N. Kostem. User's Manual for Program BOVA. Fritz Engineering Laboratory, Lehigh Univ., Bethlehem, PA, Rept. 378B.6A, March 1975.
42. C. N. Kostem. Overloading of Highway Bridges—or: A Parametric Study. Fritz Engineering Laboratory, Lehigh Univ., Bethlehem, PA, Rept. 378B.7, August 1976.

Publication of this paper sponsored by Committee on General Structures.

Importance of Redundancy in Bridge-Fracture Control

R. A. P. Sweeney, Canadian National Railways, Montreal

Because of component redundancy, riveted structures have tended to be fail-safe. It has been far less important to be aware of the limits of fatigue and brittle fracture in riveted structures than in welded structures, which are generally not component fail-safe. In the change from riveted to welded-plate girders, the safety factor protecting against brittle fracture in nonredundant load-path structures has weakened. The inherent crack stoppers at interfaces between components of riveted structures do not exist in structures that are welded or repaired by welding. Designers must therefore design, fabricators must produce, and inspectors must examine relatively crack-free structures and ensure that they will not develop large cracks during their service lives. This safe-life approach is an absolute requirement for nonredundant load-path structures. Several examples of cracked structures that have not collapsed because of redundancy are given, and the effect of welded repairs is discussed. The paper illustrates the redundancy of several simple trusses with a discussion of bridge fires. Strict application of these guidelines will force many designers to change to redundant load-path or component-redundant structures (e.g., bolted) in many instances, particularly in the short-span range, as alternatives to the additional material that may be required to avoid fracture.

Fatigue and fracture are apparently far more serious problems in welded structures (1) than in riveted structures. Part of this is because, during the long experience with riveted structures, most of the really bad details were eliminated; part is also because riveted structures have an inherent component redundancy and somewhat lower rigidity.

Just after the turn of the century use of redundant members was frowned on. Waddell (2) pointed out that the resulting ambiguity of stress distribution could lead to insufficiently designed connections or to an error in following a load to its conclusion. He also emphasized that, in checking a structure, one must follow each stress given on the stress diagram, from its point of application on one main member until it is transferred completely either to other main members or to the substructure, and see that each detail by which it travels has sufficient strength to resist the stress that it carries.

Clearly it was not possible to apply these principles to a structure in which the designer had no idea of the load path.

REDUNDANCY

Against fracture, however, riveted structures were at least internally member redundant in that most members were built up of several components (Figure 1). This component redundancy comes about because cracks do not jump from piece to piece.

The chord member of the truss shown in Figure 2 cracked on only one side of the member and carried rail traffic for some time before detection and repair. In a welded-box member the crack would have propagated all around (Figure 3).

In spite of the above principles, most trusses were multi-load-path structures. The truss in Figure 4 had its bottom chord completely severed and yet remained standing because of the alternate load paths provided by the bracing, floor system, etc. Unfortunately, the truss shown in Figure 5 did not follow Waddell's rules for adequate bracing and did not remain standing when one of its diagonals was severed by a shifting load. It had insufficient redundancy.

To demonstrate clearly that redundancy did exist in structures designed by these principles it is only necessary to examine a few bridge fires and their resulting locked-in residual stresses.

One of our trusses was subjected to a deck fire. On cooling, a gun-like explosion indicated a crack in the bottom chord (Figure 6). In a simple truss that had no other mechanism for carrying load, this fracture would have caused collapse. Over the next three weeks early spring temperature cycles caused several more of these rapid fractures. There was no live load and the dead-load stresses were rather low, so the driving force must have been locked-in residuals from the fire coupled with the relatively small stresses from temperature varia-

Figure 1. Member redundancy in a riveted structure.

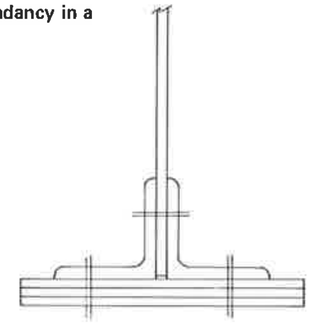


Figure 2. Cracked chord member.

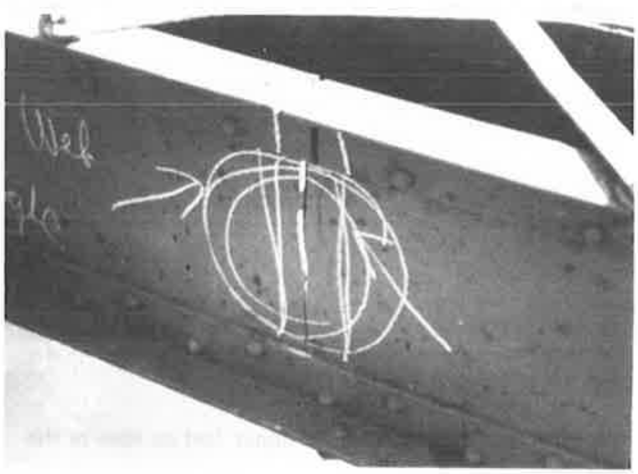
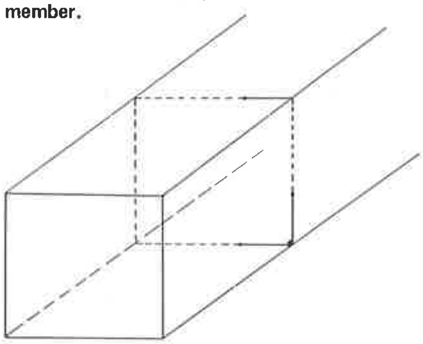


Figure 3. Crack propagation in a welded-box member.



tions. The crack shown in Figure 7 developed three weeks after the fire.

The mechanism proceeds as follows. The truss tries to expand when heated. After a certain amount of preliminary expansion, further expansion (Figure 8) is restrained by the adjacent members, bracing system, top chord, and so on. Eventually the main member yields in compression as expansion is restrained. On cooling, the member must contract the usual amount (Figure 9) but again is prevented from doing so by the other members restraining it from contracting past its original equilibrium position; this induces high residual tensile stresses.

The arch shown in Figure 10, designed with a simple pin at the center by Waddell, is about as simple as possible. Yet, under a fire, the measured residual stresses in the top chord exceeded the yield point. Again the

Figure 4. Completely severed bottom chord.

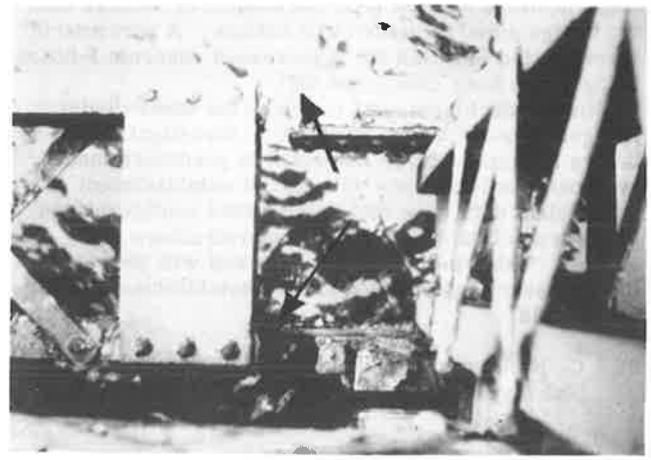
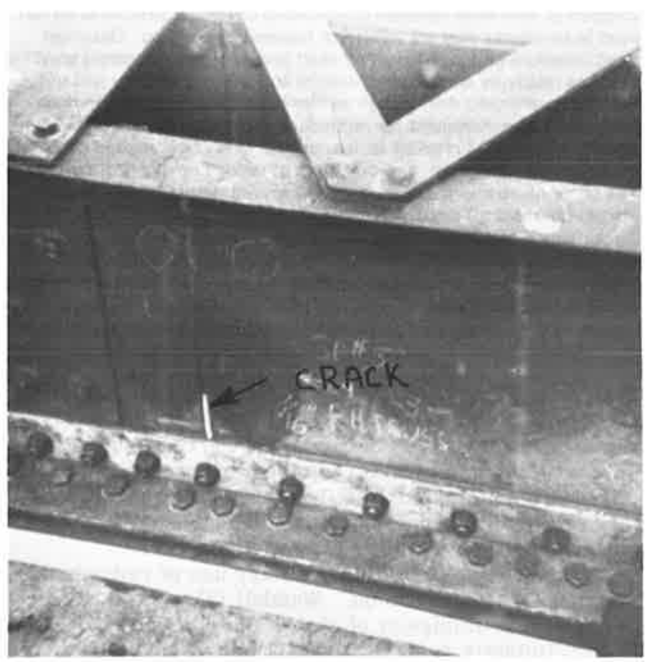


Figure 5. Result of inadequate bracing.

Figure 6. Crack caused by cooling after deck fire.



internal redundancy preventing large movements was the cause.

Truss-like structures were never designed for the gross movements induced by fire, so it is not surprising that they have problems in fires. Nevertheless, fires prove that these structures do have considerable re-

Figure 7. Crack developed three weeks after deck fire.

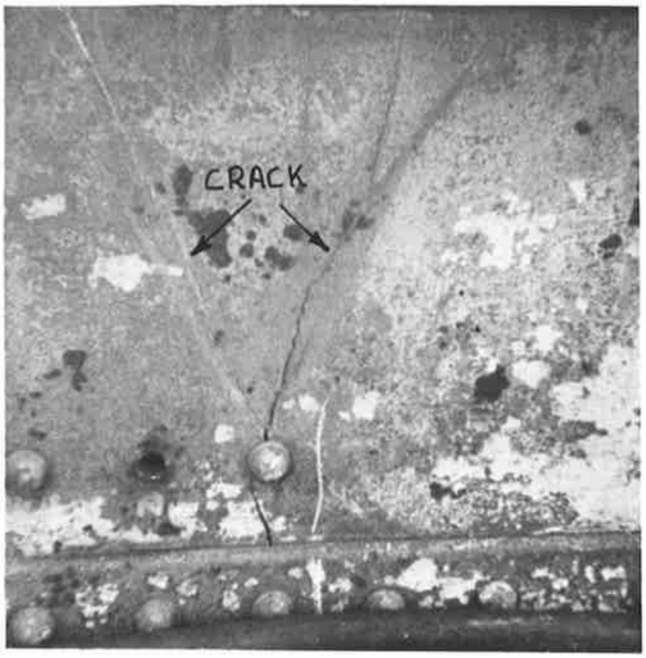


Figure 8. Expansion with and without restraint.

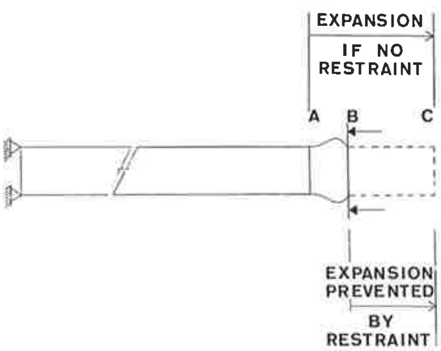
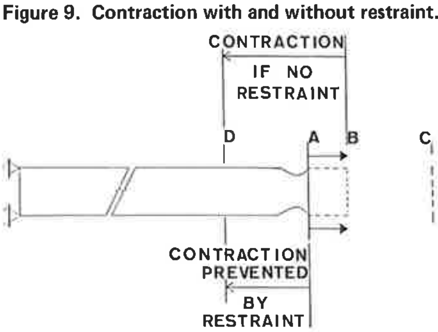


Figure 9. Contraction with and without restraint.



dundancy or alternate load paths that will prevent collapse by brittle fracture.

Both riveted and welded structures can be made redundant by building in multiple-member load paths, as in a multibeam bridge. Bracing systems can also add to redundancy, and riveted structures are internally member redundant in that most of their members are made of built-up components.

However, if one considers the deck-plate girder

Figure 10. Waddell arch under deck fire.

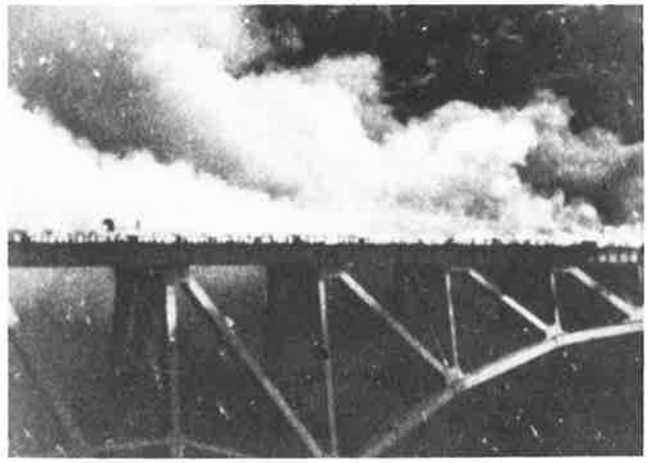
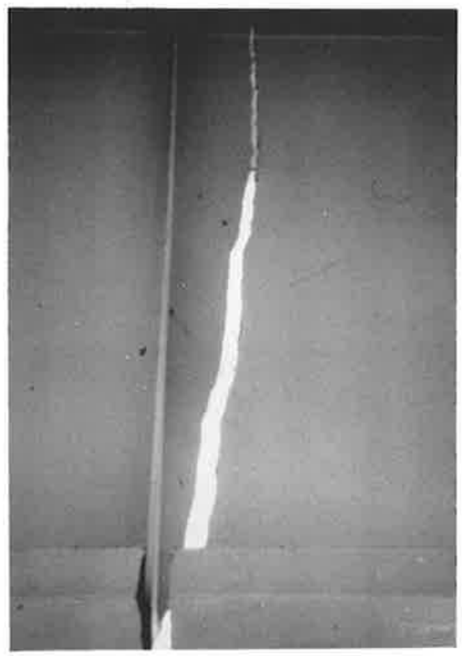


Figure 11. Crack becoming a fracture.



bridge, which on railways is generally a two-girder system, there is very little structural redundancy and virtually no component redundancy in a welded structure.

In this type of structure, if a crack is initiated by an accidental impact or by a nick fabricated in it or caused by some less than ideal detail, it may run until the structure fractures (3) (Figure 11). One must depend on what little structural redundancy remains to carry load. 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.

Furthermore, in welded structures, if a crack starts to run (Figure 12) in a weld, it will keep going until it runs out of material, weld, or driving stress. Unfortunately, the driving stress may be the yield-point tensile residual stress left by the welding process.

In a riveted structure whose rolled components do not have this nearly constant, high residual stress field, the

Figure 12. Crack growth.

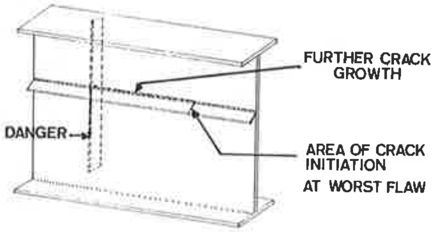
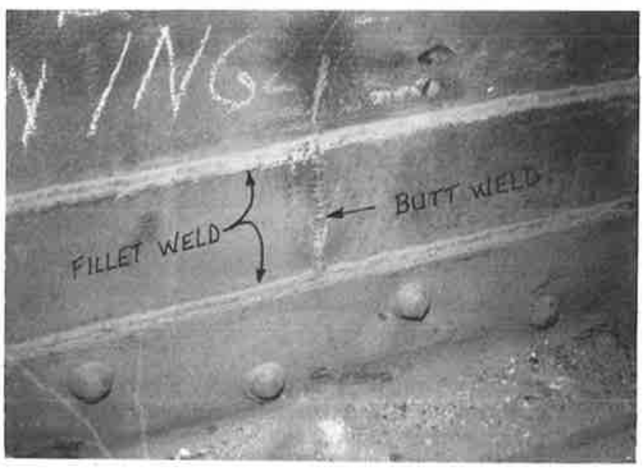


Figure 13. Girders with a Pratt truss.

Figure 14. Butt and fillet welds.



crack will run out of energy as the primary load-carrying stress field is diminished. In the typical riveted structure, a crack, from whatever source, will propagate only within the component that has 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 opposite flange angle. Another crack may develop, but this will take time and further application of load. Inspection should reveal the initial crack so that necessary repairs can be made before a serious problem develops. The probability of two cracks initiating together in such a component has been calculated to be virtually insignificant (4). This type of component redundancy permits much greater inspection intervals and therefore more time to schedule repairs. It often al-

Figure 15. Crack propagated into the web.

lows repairs to be deferred until other items combine to make it worthwhile to send in a repair crew. In several instances in my practice, this time interval has been on the order of years.

REPAIRS

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

In the late 1960s, it was decided to repair the corrosion that occurred in deck-plate girders at the web-bottom flange junction on a number of our structures. The procedure was to sandblast the bottom foot or so of the web and to weld patch plates over those areas where corrosion was worst. This was done in one season on a production basis on all the structures between two major cities. The girders shown in Figure 13 rated E90+ in new condition and with the web holed along the web-flange interface; with a Pratt truss mechanism to carry shear, they still rated E75 under fairly severe corrosion.

If the welds had all been of American Welding Society (AWS) quality and if the patch plates had been run out to areas of sufficiently low stress range, the detail would have been theoretically adequate, although it would have been essential to be able to make shop-like field welds.

Because of the production operation, patch plates varied in length and were butt welded as required (Figure 14). After 10 years these welds started to crack under very low stress ranges. The maximum measured stress range was 36.2 MPa (5250 lbf/in^2) with a mean peak per train of 31.92 MPa (4630 lbf/in^2) with a sample of 12 trains. The mean plus two standard deviations of these peak stress ranges per train was 36.47 MPa (5290 lbf/in^2). There was little if any apparent dynamic augment.

The welds were not of AWS quality, which was not surprising for field welding of this nature. Nevertheless, it probably means that they would have cracked after 15-18 instead of 10 years. The cracks (Figure 15) in these welds were very hard to detect and in fact were not detected until they had propagated from the patch-plate welds into the web.

The crack path was as follows:

1. Cracks initiated at a flaw; some were interior but most were surface flaws;
2. Cracks grew (Figure 16) upward and downward in

Figure 16. Direction of crack propagation.

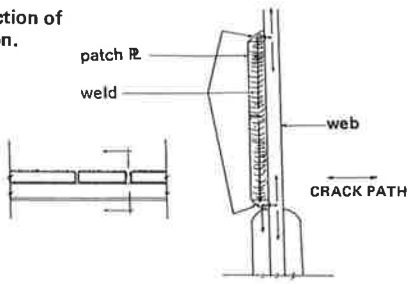
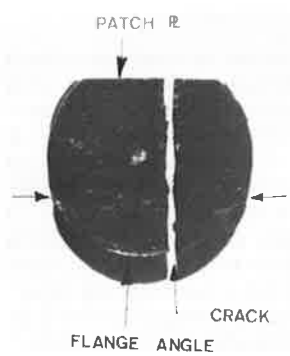


Figure 17. Core cut for plug.

Figure 18. Plug showing welds.



the weld in those welds that did not have full penetration and horizontally into the web first for those that did;

3. Growth was from the level of the horizontal fillet welds into the web and bottom flange and left multiple cracks;

4. These multiple cracks then grew.

Several plugs were taken to confirm the above process. From the location shown in Figure 17, a core 76 mm (3 in) in diameter was cut out with a hole saw. The core was then cut through the upper leg of the bottom flange angle and patch plate and then web to patch plate and web and patch on the other side. The plug (Figure 18) shows the vertical (just right of center) and horizontal (just below center) welds. The left piece in Figure 19 shows the patch plate, which was above the leg of the flange angle, that had the crack in it from the inside. The center piece is the web corroded just above the horizontal line. The right piece is the opposite-side patch plate and bottom flange angle and shows its crack

Figure 19. Patch plates and corroded web.

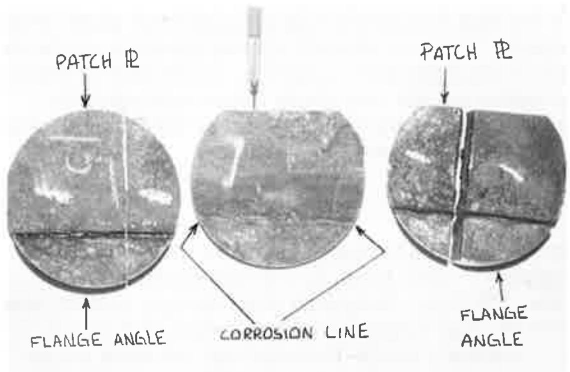
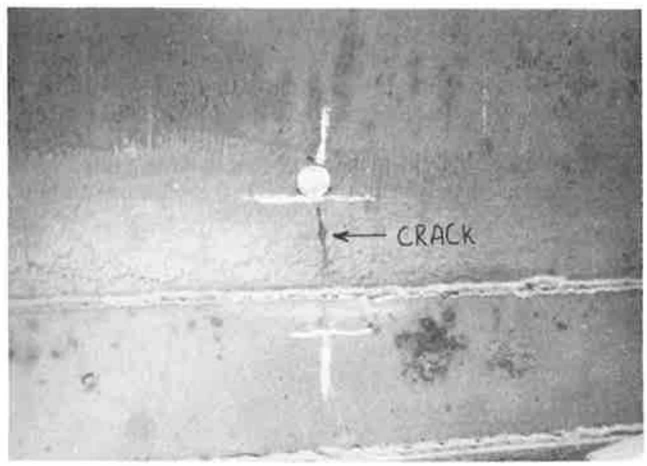


Figure 20. Inside of girder before core was drilled.



at a different position than the first one (also viewed from the inside).

Figure 20 shows the same location, before the core was drilled, from the inside of the girder. Note that there is no butt weld on the inside at the location of this crack, which propagated from the opposite side.

Four crack fronts initiating from the same crack have been illustrated. Depending on the degree of penetration, 10 or more crack fronts could theoretically develop. Therefore, the repair procedure cannot consist solely of repairing the visible cracks, because there may be other masked cracks. Component redundancy will permit a delay in making a full repair.

A permanent repair involves removing all the patch plates and then the welds, inspecting underneath for cracks, stopping the cracks, and putting on bolted patch plates. A new span could be fabricated and the old spans removed one at a time, repaired on the ground, and leapfrogged ahead until all spans are repaired. The alternative is frequent inspection.

If a welded repair is to be used, it must conform to current code requirements. That is, the welds must meet AWS standards and the plate ends must be in low stress range areas.

If the original repair was bolted or riveted, a crack that had initiated would not have transferred to the web or flange angles. The major problem with welded repairs to a riveted or bolted girder is that they destroy the initial component redundancy of the girder (Figure 16). But it is very tempting to use welded repairs. If this is done, the welds can transfer any crack to the material to which it is welded. If this could lead to a catastrophic failure, it should obviously not be permitted.

In the aircraft industry crack stoppers are used. They 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 bypassed.

WELDED STRUCTURES

In current all-welded bridge structures there are no crack stoppers. This leads to a requirement of safe-life design. That is, designers must design, fabricators must produce, and inspectors must ensure that structures are relatively crack-free and will not have large cracks during their service lives. This is not impossible, but it is difficult and requires special attention to details, fabrication, and inspection.

Designers now have guidance on typical details (1) in the current Association of American State Highway and Transportation Officials (AASHTO) and American Railway Engineering Association (AREA) codes; AWS goes a long way for the inspection teams, and fabricators must be far more careful than they have been with riveted or bolted structures.

Remember (Figure 11) that in this welded-plate girder a single crack was sufficient to split the member in half, whereas in a typical riveted structure this generally would not happen. Because of component redundancy, riveted structures have tended to be fail-safe. It was therefore far less important to be aware of the limits of fatigue and brittle fracture in these than in welded structures, which are generally not component fail-safe. In changing from riveted to welded-plate girders, there was a reduction in the safety factor that protected against brittle fracture in nonredundant load-path structures. AASHTO has recognized this in its current fatigue provisions in that it permits smaller stress ranges for nonredundant load-path structures. For example the permissible stress range for an "E" detail, say the end of a thin cover plate, at run out is 34.47 MPa (5000 lbf/in^2) in a redundant load-path structure and only 17.24 MPa (2500 lbf/in^2) in a nonredundant load-path structure.

AREA has not recognized this yet (1979) in its manual, although a proposal to do so has been made. This is most unfortunate, because single-track railway bridges tend to be very narrow and readily lend themselves to typical two-girder solutions. Designers, fabricators, and inspectors should be cautioned to be conservative with nonredundant load-path structures.

One way of ensuring more fail-safe nonredundant load-path structures is to insist on much tougher steels. This is done in the nuclear field and to some extent in Canadian railway bridges. This is not necessarily the most cost-effective solution in bridges at this time (1979), although it has prevented collapse in a number of instances.

The fracture-toughness requirement for bridge steels should be relegated to a position of secondary importance once the particular candidate material has been shown to possess an adequate level of fracture toughness (5).

Figure 21. Damage caused by backhoe.

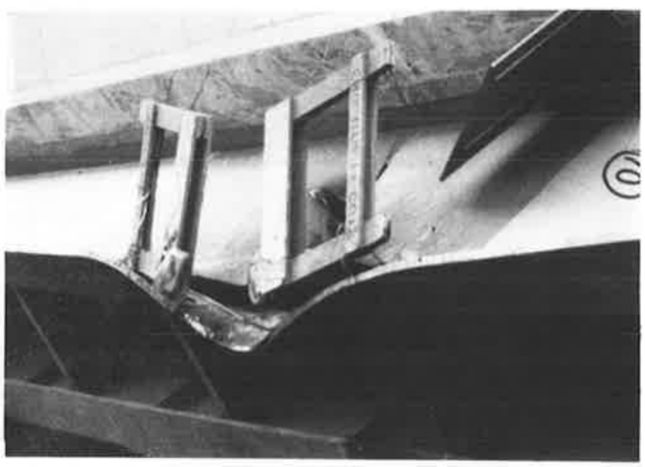
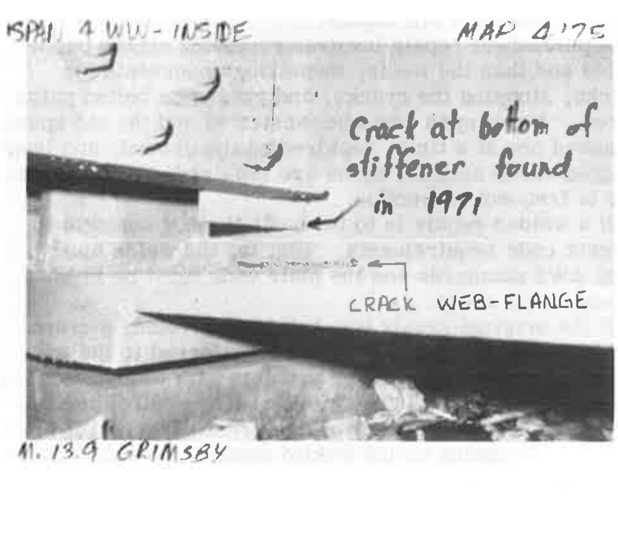


Figure 22. Crack caused by fatigue-related failure.



EXAMPLES

In order to emphasize the above discussions, a few illustrations of failures where redundancy has saved welded structures or where component redundancy has saved riveted structures should be instructive.

A rather striking example is the multibeam welded structure with a composite deck that was hit by the top of a backhoe (Figure 21). Although several girders were badly damaged, the structure did not collapse under train traffic. Several trains are known to have crossed on the adjacent track after the accident. Imagine what could have happened if that had been a two-girder noncomposite welded system.

A skewed multibeam composite structure (Figure 22) cracked due to a fatigue-related failure. Throughout the repairs, which were delayed for over a year, regular train traffic was permitted. This would not have been possible on a nonredundant load-path system.

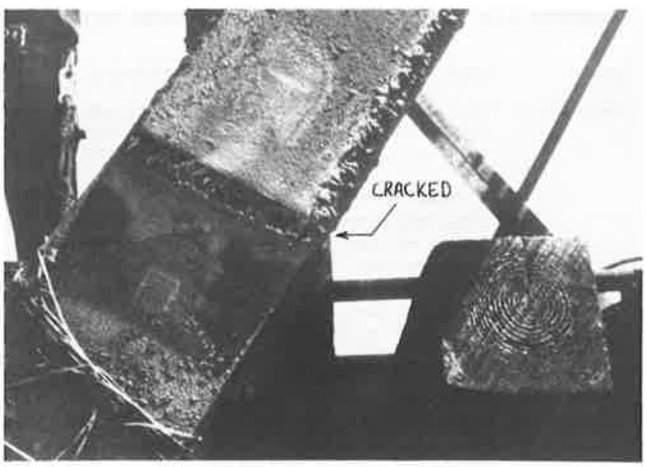
The structure in Figure 23 cracked from torsional fatigue caused by constant train braking. Although the floor beams were not welded, there was no component redundancy as the beams were rolled. Nevertheless, the redundancy of the deck system permitted regular train operations for over a year even though four adjacent floor beams failed.

The type of repair shown in Figure 24, the addition of welded plates to an eyebar member, could be the source of a disaster in a nonredundant load-path structure.

All four of these examples of transverse welds were found to be cracked after about 15 years of service. 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.

Figure 23. Crack caused by torsional fatigue.

Figure 24. Repair of eyebar member.



OTHER POINTS

There are a number of other points that should be mentioned to illustrate why riveted or bolted bridges seem to stand, while seemingly better-welded bridges occasionally crack and fail. For example, secondary displacements could be tolerated in riveted structures because of component redundancy and the ability to give under load. These same secondary displacements have caused failures in welded structures (6) for converse reasons. Nonredundant load-path structures that have not been designed for displacements could experience catastrophy.

In addition, one must remember that the design rules are valid only if all welds meet AWS standards. Inspection then becomes more critical in a nonredundant load-path structure. Designers need to know the level of quality control their inspectors must exercise.

Obviously, two-girder systems make bracing much simpler, and this advantage must be balanced against the possibility of collapse from brittle fracture. Basically, lower stress levels must be ensured. Strict application of these guidelines will force many designers to change to redundant load-path or component-redundant structures (say, bolted) in many instances, particularly in the short-span range, as an alternative to the additional material that may be required to avoid fracture.

In the last analysis, economics should be the deciding factor, which, nevertheless, must be based on the economic comparison of adequate structures and not on a comparison of structures similar to those experiencing difficulty with fatigue or brittle fracture now.

One designer, when first introduced to the limiting values of a "C" detail at the bottom of a stiffener, remarked that the lower stress-range level would require more than double his design flange thickness. The design stress range for this redundant structure was 186.16 MPa (27 000 lbf/in^2) on a short-span element. The AREA permissible stress range was 82.74 MPa (12 000 lbf/in^2). The life predicted by using the commentary on the AREA code (1, 7) would be 4.7 years. A similar structure, where the AREA design spectrum and the actual load spectrum were found to be very close, cracked in five years.

In case it is thought that this represents a new phenomenon, consider that in 1891 Waddell (8) found he could no longer compete with certain designers, because to ensure against collapse he had to make his structures too heavy. As a result of a considerable number of failures, the profession became aware of the problem and resolved it by the beginning of this century.

CONCLUSIONS

1. Considering that it is not practical to build welded structures that have no flaws or to return to solely riveted or bolted construction, designers can make use of redundancy, component or structural or both, to assist in fracture control. This is a fail-safe approach.

2. On the other hand, the designer of a welded nonredundant load-path structure must ensure that there will not be any brittle fracture during the life of the bridge. This requires that the fatigue mechanism not generate cracks large enough to cause sudden rupture and that the strictest attention be paid to details and to lower stress levels than those in the current AREA code. Levels should be similar to those recommended by AASHTO. The designer must be certain to account for all potential loads due to primary and secondary displacement and to reasonably predict accidental and erection loads.

Although this has not been emphasized in this paper, such a design requires certainty as to adequate toughness of the material to overcome the flaws and unexpected events to which it will be subjected. The current codes give adequate guidance in this respect.

3. The designer must be sure that the inspector can detect all flaws larger than those assumed by the code writers in setting their permissible stress levels and that the fabricator realizes that large defects are a certain cause for rejection.

This safe-life approach is an absolute requirement for nonredundant load-path structures.

REFERENCES

1. J. W. Fisher. Bridge Fatigue Guide. American Institute of Steel Construction, New York, 1977.
2. J. A. L. Waddell. De Pontibus. Wiley, New York, 2nd Ed., 1914.
3. R. A. P. Sweeney. Some Examples of Detection and Repair of Fatigue Damage in Railway Bridge Members. Paper presented at the 57th Annual Meeting, TRB, Jan. 1978.
4. W. G. Byers. Rating and Reliability of Railway Bridges. In Methods of Structural Analysis, Proc., ASCE Specialty Conference, Madison, Wisconsin, 1976, pp. 153-170.

5. J. M. Barsom and S. R. Novak. Subcritical Crack Growth and Fracture of Bridge Steels. NCHRP, Rept. 181, 1977.
6. J. W. Fisher. Fatigue Cracking in Bridges from Out-of-Plane Displacements. Canadian Journal of Civil Engineering, Vol. 5, No. 4, 1978, pp. 542-556.
7. Committee on Steel Structures. Revision of Manual. AREA Bull. 660; Proc., Vol. 78, Nov.-Dec. 1976, pp. 77-90.
8. D. C. Jackson. Railroads, Truss Bridges, and the Rise of the Civil Engineer. Civil Engineering, Vol. 47, No. 10, Oct. 1977, pp. 97-101.

Publication of this paper sponsored by Committee on Steel Bridges.

Bridge Design Procedures Based on Performance Requirements

Geerhard Haaijer, Charles G. Schilling, and Phillip S. Carskaddan, Research Laboratory, United States Steel Corporation, Monroeville, Pennsylvania

In 1977 the American Association of State Highway and Transportation Officials (AASHTO) introduced redundancy as a parameter in bridge design, specifically by making allowable fatigue-stress ranges a function of a structure's redundancy. However, recently proposed design procedures would allow redundancy and fatigue to be handled more directly; they would bring design assumptions in closer agreement with actual behavior. This paper reviews these new design procedures from a load-factor approach. At each factored load level, structural performance requirements are defined and then limit-state criteria are established to satisfy these requirements. The AASHTO limit states and the recently proposed limit states are compared, because both are intended to satisfy the same performance requirements. At service load, a new fatigue design procedure reflects actual conditions. At overload, the ability of a redundant structure to shake down is recognized. At maximum load, the strength of a redundant structure is computed with plastic design methods. And, finally, a preliminary procedure for fail-safe analysis that would apply only when fatigue governs is introduced. An engineer following these new procedures is encouraged to include redundancy in bridges for more rational design.

In 1977 the American Association of State Highway and Transportation Officials (AASHTO) introduced redundancy as a parameter in determining allowable fatigue-stress ranges for steel bridges (1). A new table of allowable stress ranges was added for nonredundant structures to provide increased safety by requiring a shift of one range of loading cycles for fatigue design, thereby reducing allowable stress ranges. However, research into the behavior of steel structures has resulted in new design procedures that handle redundancy and fatigue more directly. These methods close some of the gaps between design and actual conditions. Similar methods are appearing elsewhere: In California the design method is correlated with permit policy (2), and in Ontario the new highway bridge design code relates limit-state design to actual loadings (3).

This paper describes four new design procedures that use the same approach as that currently used in the AASHTO load-factor design (LFD) specifications (4). Before describing these methods, it will be helpful to review performance requirements.

STRUCTURAL PERFORMANCE REQUIREMENTS

For any loading, a designer understands the behavior expected of the bridge. This behavior may be stated in terms of structural performance requirements associated with various load levels. The requirements for the AASHTO LFD method are shown in the table below.

Source	Load Level	Requirement
AASHTO	Service load [D + (L + I)] (dead plus standard vehicles)	Provide fatigue life, control elastic deflections, limit concrete cracking
	Overload [D + 5/3(L + I)] (dead plus occasional permit vehicles)	Control permanent deformations possibly impairing ride quality
	Maximum load {1.3[D + 5/3(L + I)]} (increased dead plus few exceptional vehicles)	Resist load
Proposed	Fatigue load [(L + I) 50/72], or by study (effective truck)	Provide fatigue life
	Fail-safe load [αD + β(L + I)] (only checked if a detail has a design life less than a specified value, e.g., 100 years)	Resist load when one element is separated

To ensure that a bridge behaves according to the stated structural performance requirements, a designer may establish limit states for each requirement. A limit state is a constraint such that if the structural behavior exceeds the constraint the performance requirement may not be satisfied. Various limit states may be selected to satisfy the same performance requirements. For example, the AASHTO LFD and working-stress design methods both result in bridges that behave satisfactorily. However, a standard by which the merit of a limit state can be judged is how close that limit state is to actual behavior.

In the following section, four recently proposed design methods and their limit states are described. The methods are grouped under four load levels: the three AASHTO factored load levels and a proposed load level.

PROPOSED DESIGN METHODS

Service Load

When subjected to service loads, a structure's primary performance requirement is that it have an adequate fatigue life. AASHTO currently achieves this performance requirement by limiting allowable stress ranges.