COLLAPSE OF ASHITABULA BRIDGE ON
DECEMBER 29, 1876

By D. A. Gasparini, Member, ASCE, and Melissa Fields

Abstract: The design, fabrication, erection, and performance of an iron railroad bridge built in 1863-65 at Ashitabula, Ohio, is examined. The bridge collapsed after 11 years of service, very likely due to fatigue and brittle fracture at a flaw in an iron casting. The bridge was built by a master builder/entrepreneur, using processing procedures developed for wooden Howe trusses. At the time, structural analysis was an evolving art, the capacity of slender compressive elements was still an issue, and design specifications did not exist and fatigue was largely an unknown phenomenon. The failure was investigated extensively by engineers. Most focused exclusively on static strength issues; only one observed the flawed casting. The failure bolstered the call for consulting bridge engineers and standard design specifications. It brought in focus the issue of reliability of iron castings and by 1888 Cooper's specifications explicitly forbade use of cast iron in any part of a bridge structure.

Introduction

The objective here is a structural reevaluation of the design, fabrication, erection, and performance of an iron railroad bridge erected over a stream at Ashitabula, Ohio, in 1863-65. To provide a context for the discussion, it is important to review, briefly, the history of the Howe truss system used, the state of the art in structural analysis at the time, and contemporary knowledge of strength of materials.

In 1830, Stephen Harriman Long received a patent for an all-wood bridge truss; very likely, Long's system was the first parallel chord truss used for carrying railroad loads. A feature of the truss was that the wooden diagonals were prestressed in compression by means of wedges. An advantage of the prestressing was that both diagonal braces become active in resisting the applied load without having to detail a tension connection for the counter-brace. Also, some wood shrinkage could occur without concomitant loosening of the connections. In the 1840s, William Howe, his brother-in-law Amasa Stone, Azariah Boody, and Daniel Harris developed and marketed a truss that was the same as Long's truss except that the vertical elements were iron rods and the prestressing was done by "screwing up" the nuts on the vertical rods. The Howe truss was much easier to construct and prestress; it became the dominant truss type for transitional bridges made with wooden diagonals and vertical iron rods. From about 1845, iron versions of the Howe truss were also built. J. G. James (1980) has noted that Amasa Stone and D. 1. Harris built an iron Howe truss of 50-ft span for the Boston and Providence Railroad. In addition, Simmons (1989) has described an Iron Howe pony truss built by Stone in 1850 for a crossing of the Ohio Canal.

In 1836, Stephen Harriman Long published his booklet: "Description of..."
Tomlinson testified that the bridge was laid out taking it's bearings from the old truss. He then later testified that the bridge would still have the same form as shown in Figure 7.1 without the truss, and he described the bridge as being 60 m long. It is uncertain whether Stone or Tomlinson specified the form of the bridge, or if the bridge was designed without consideration for the trusses. Some important features of the design are:

1. The compressive diagonal and the top chord of the bridge consist of a group of I beams in parallel, not continuously connected. The second panel point on the top chord must transfer the axial force from the two I beams to the compression. [See Figure 3.14 of the report.]

2. The beams forming the top chord rest directly on the top chord, thus placing the top chord in flexure. The rails are placed such that a train essentially loads only one of the two trusses.

3. The compressive diagonal and the top chord of the bridge consist of a group of I beams in parallel, not continuously connected. The second panel point on the top chord must transfer the axial force from the two I beams to the compression. [See Figure 3.14 of the report.]

4. The rails are placed such that a train essentially loads only one of the two trusses.

Sizing all parts of the bridge, of course, requires definition of design loads and analyses to determine the effects of the loads on all elements of the bridge. The loads considered in Tomlinson's design were:

- Dead load: the weight of the bridge itself, including all materials used.
- Live load: the weight of the train, including the weight of the train and its equipment.
- Wind load: the force exerted by the wind on the bridge.
- Earthquake load: the force exerted by an earthquake on the bridge.

Tomlinson testified: 'We have to make the bridge strong enough to support the train. We must also consider wind, earthquake, and live loads. We must also consider the effects of the train's motion on the bridge.'
wood diagonals that were less slender. If it is assumed that the braces were
not rotationally or translationally restrained anywhere along their length
(although counter braces were used) and that the ends are pinned, then
a stress of 55.2 MPa (8,000 lb/sq in.) would buckle the I sections
used. Tomlinson proposed to increase the strength of the compressive braces
by "riveting plate to them," but Stone was "wrathy, and would not hear"
(Report 1877). In addition, apparently the I beams actually supplied were
somewhat undersized; Tomlinson testified that the sections of the braces
"were not what the drawings represented; and when they came to be turned
out from the mill they didn't hold full size, as intended" (Report 1877).
Because of these conflicts, Tomlinson either resigned or was discharged by
Stone.

The exact details of the initial design of the compressive diagonal braces
and the angle blocks are uncertain. Fig. 3(a) shows a likely detail; that is,
a brace consisting of four I beams with the flanges in a vertical plane. Some
luggs were cast on the iron angle block to prevent movement of the diagonals,
especially during erection.

Neither Tomlinson nor Stone testified in detail on the issue of bending
in the top chords, nor on any other loading condition such as wind or
snow or the forces induced by tightening the nuts on the vertical elements.
**Fabrication**

The castings and forgings for the bridge were made at the Lake Shore shops in Cleveland under the direction of Albert Congdon, master mechanic. The iron was bought from the “Newburg mills” (Report 1877). The I sections were rolled by Stone, Chisholms, and Jones.

Congdon testified that “the plans were handed to me by Mr. Tomlinson” and that “Mr. Tomlinson made the drawings or working drawings for the angle blocks” (Report 1877). He further testified that “I knew Mr. Tomlinson approved of the blocks before they were moulded.” Tomlinson agreed: “Yes, sir; I gave the patterns and the length of every part of it, for the mechanics to work by” (Report 1877). Tomlinson supervised the entire fabrication: “I have only a kind of indistinct consciousness that the pattern makers were making some patterns which were not finished, and which I didn’t see what they were. The surface of the angle blocks were not planed, and there were some little things not finished; but it was nothing essential” (Report 1877). Tomlinson testified: “The bottom chords of the bridge, the tension members, were as good work as ever I saw made” (Report 1877).

As stated previously, the angle blocks for the first two panels were probably detailed to accept four I beams and to provide some restraint. According to Tomlinson, some of the I beam sections as rolled, were undersized.

**Erection and Test Loading**

Tomlinson had resigned and Stone chose not to give the responsibility for the erection of the bridge to Charles Collins, the chief engineer for the railroad. Stone himself supervised the erection, which was carried out by A. L. Rogers, a carpenter who built and repaired wooden bridges for the railroad. When asked, “Had you ever had any experience in iron work or the raising of iron structures before that?” he testified, “not in the least” (Report 1877).

Prior to erecting the iron bridge, Rogers built the wooden “false work to raise the bridge upon” (Report 1877). A critical aspect of the false work was the built-in camber. Rogers testified that he followed Congdon’s suggestion that it should be between 12.7 cm and 17.8 cm (5 and 7 in.). However, during a visit to the site, Stone remarked that “the bridge is not designed to have so much camber as that. The camber of the bridge is one-half inch to the panel, or a shade less,” whereupon Rogers “changed its bearings back, planed them, and jacked them off, so as to get them as near right as I could, and then went to work to put it on, so that the camber must have been three and a half inches” (Report 1877).

The 8.9-cm (3.5-in.) camber, which was smaller than the one used by Tomlinson to detail member lengths, probably caused the first erection problem. As Rogers testified, the “members of the upper chord were too long; they wouldn’t go in between the lugs on the angle blocks, because they were set in these braces before that upper chord was put on.” There is some uncertainty regarding what was done to fit the chords. Certainly the chords were shortened, although Rogers testified that a much more difficult job was also done, reducing the thickness of the vertical lugs (Rogers may have been confused with the later removal of the lugs used for positioning the diagonal braces). The final step in the erection prior to removal of the false work was the “screwing up” of the vertical rods to pretress the truss. It is uncertain how much prestressing was achieved, but with the
shortened chords the bridge deflected 6.4 cm (2.5 in.) below the horizontal when the false-work blocks were slowly removed and the dead load became active. The removal of the false work was halted. At this point, Stone instructed that the chord lengths were to be increased to their original values by using shims (the shims probably fit because Rogers increased the camber of the false work to the original intended value of 15.2 cm [6 in.]). At this stage, everyone wanted a “tight” bridge. In fact, Rogers testified that “I tightened the vertical rods so tight that in one or two instances I had buckled the braces by drawing the chords together” (Report 1877). Even if Rogers had released the nuts until the buckles were no longer visible, it is likely that some diagonals were prestressed close to their buckling load. Given such initial prestress forces, when the false-work blocking was removed for the second time, some diagonals began buckling under the action of the dead load. Removal of the false work was again stopped.

A this stage, Congdon, Collins, and Stone all recall stating that the direction of the diagonals should be changed. Congdon testified that he told Stone, “if those braces were turned up edgewise, says I, they would have more than twice the carrying weight without adding any extra weight to the bridge.” At the request of Rogers, Collins also spoke with Stone: “I said to him, sir, that the braces were in wrong; they got them in flat, instead of on edge, in my opinion.” Stone testified: “As the bridge was finally erected, an error was made by the parties who were raising it . . . . they put in the braces flat-wise, horizontal instead of vertical.”

More importantly, Stone must have reflected on Tomlinson’s objections and decided to add I beams to the diagonal compressive braces in the end panels. There is some uncertainty regarding exactly how many, where and what size I beams were added. However it is most likely that two I beams were added to the braces of the first two panels and one I beam was added to the brace in the third panel, as shown in Fig. 4.

To fit the additional I sections, now turned as shown in Fig. 3(b), two important modifications had to be made. One was to remove the angle block lugs intended to restrain the diagonal braces and another was to “chip” some ends of the braces in order to clear the vertical iron rods. This meant that the ends of the diagonal elements would, in some cases, not have a “square” bearing and that there was no mechanical restraint—except friction from the axial forces—to prevent movement of the diagonals (this was largely true in wooden Howe trusses). After the modifications were made, the bridge was again prestressed. Rogers testified, “After the iron was got ready, the bridge was then set on its own bearing and stood so; you couldn’t see it settle one hair. There was no buckle to the braces after that I ever noticed.”

A load test was then performed; the Ashubula Sentinel (1877) reported that the bridge was tested “by running three engines (instead of six) at a fair speed and then standing them on it. The structure was depressed five-eighths of an inch; when the engines moved off, the bridge sprung back three-eighths of an inch leaving a sag of one-fourth inch. The work was then pronounced finished.” The actual weight of the test train was uncertain, although it is likely that 267-kN (60,000-lb) engines were used. The bridge was then placed in service.

**SERVICE AND INSPECTION**

Only one track was initially installed on the bridge, but Collins testified that a second track was added soon after. The inspection record is vague. Congdon testified that he inspected the bridge two years after completion: “I examined it thoroughly, and a day or two afterwards, I told Mr. Stone, at his office, I told him I had examined the bridge and it looked very well; I didn’t discover anything but one brace, and that was but a trifle out, I could just jar it; it was probably loose enough—Oh! you couldn’t slip a bank bill between the two wedges; it was an outside brace, and in about the third panel from the east end, as near as I can recollect. As I knew Mr. Rogers had the superintending of the erecting of the bridge, I told him of it. My idea was that he would go perhaps and turn that outside nut a little—it didn’t require but a sixteenth turn, probably.”

Charles Collins, as chief engineer, had the responsibility for inspection with a department headed by G. M. Reed. Collins said that there was no specified maximum time between inspections and gave the following testimony regarding the last inspection:

Q: When did this Reed, who is charged with this duty, make a report to you in relation to the bridge?
A: In September, I think.
Q: What report did he make then?
A: It was right.
Q: It was in good condition?
A: Yes.
Q: Was that report made in writing, or verbal?
A: Verbal.

Gustavus Polson, a train engineer who survived the collapse, did testify that “I noticed in passing over it at times a snapping as if the joints were settling together. I hear that snapping, so to speak, but did not feel it. It was not a rattle or continuation of sound, but it was a snap, and when heard it was over with. I never, before that night, noticed any lateral motion to the bridge in passing over it” (Report 1877).

No other incidents are known regarding the Ashubula bridge. It apparently performed satisfactory service for over 10 years.

**COLLAPSE OF ASHTABULA BRIDGE**

At about 7:30 p.m. on December 29, 1876, in the midst of a severe snowstorm, a train pulled by two locomotives was crossing the bridge, heading west toward the Ashtabula station, about 152 m (500 ft) ahead, at a speed “from twelve to fifteen miles per hour” (Report 1877). As the first
 locomotive was about to complete the crossing, the bridge began to fail. The first locomotive was able to pull to safety onto the west abutment but the second locomotive, its tender, and 11 cars fell 20 m (65 ft) into Ashtabula Creek. The fall and the severe fires started by the coal heating stoves in the cars caused over 80 deaths. It was a tragedy that genuinely shocked the nation.

The railroad acted quickly to salvage the iron and restore service. Most of the bridge was removed to the railroad’s Cleveland yards and on January 18, the Ashtabula Sentinel reported that a wooden Howe replacement bridge was completed and carrying traffic.

**INVESTIGATIONS**

On January 12, 1877, the Legislature of Ohio resolved “that a joint committee be appointed . . . to investigate the cause or causes of the recent accident.” The Joint Committee and their engineers, Benjamin F. Bowen, Thomas H. Johnson, and John Graham, reported to the legislature on January 30, 1877.

The engineers estimated that each truss carried a dead load of 18.4 kN/m (1,260 lb/ft) and an equivalent live load of 29.2 kN/m (2,000 lb/ft) from the train. The snow load was not estimated and the wind was judged to be small, causing a lateral pressure of 240 Pa (5 lb/sq ft) [this agrees with the statement of another engineer, W. S. Williams, who stated that: “Gen. Meyer, Chief Signal Officer of the United States Army, says that between eight and nine o’clock p.m., the velocity of the wind between Erie, PA and Cleveland, Ohio was from twenty-four to fifty-four miles per hour” (Report 1877)]. The engineers estimated that such a pressure would cause an “increase in the vertical force on the south truss by 2%” (Report 1877); it was also estimated that the south truss bore about 95% of the total weight of the train. That weight, equivalent to about 29.2 kN/m (2,000 lb/ft), was judged to be an ordinary load that the bridge commonly carried. Using the estimated dead and live gravity loads, the engineers determined the axial forces in all truss members. The “counter” braces were assumed inactive and no estimates of axial forces from prestressing or of bending moment in the top chords were made. The engineers then conservatively estimated the axial capacity of each member. For the compressive braces, they candidly admitted: “If the experiments upon the strength of these braces, which were contemplated by your committee, had been carried out, the value to be assigned to the bearings would have become a matter of certainty instead of opinion, and we regret they were not made, so that the results could have been embodied in this report” (Report 1877). It is difficult to estimate the strength of a compressive element because the translational and rotational restraints at the ends and at the middle are difficult to quantify and because the eccentricity with which the axial force is applied is unknown. They found that the ratio of the estimated member strength to the computed member force (the “factor of safety”) varied markedly from member to member. But in no case was this ratio estimated to be smaller than one, even with conservative estimates of member strengths.

The characteristics of the fallen bridge showed “conclusively that the failure occurred in the second and third panels of the south truss” (Report 1877), but the engineers were unable to pinpoint the one member (either the top chord or the compressive brace) that initiated the failure. In an offhand way, they concluded: “But insomuch as both members were weak, and both were involved in the break, it is of little importance which member took precedence in the failure. The factors of safety throughout the compression members were so low that failure must have followed sooner or later” (Report 1877).

Numerous criticisms of the design were expressed:

- **Howe System.** It “could be made safe” but was “excessively heavy.”
- **Compressive Diagonal Braces.** The 1 beam sizes were “promiscuously mixed.” The separate I beams were not continuously tied together. The end bearings were imperfect. There were no positive mechanical connections between the braces and the angle blocks to prevent movement of the ends of the braces.
- **Compressive Top Chords.** The I beam sizes were again mixed up and not continuously tied together. The chords were placed in bending by the train loads. The chords were braced laterally only at every other panel point.
- **X Bracing.** The vertical x-bracing between trusses and the x-bracing in the plane of both the top and bottom chords were inadequate.
- **Angle Block Castings.** The vertical lugs on the angle blocks should have been continuous.

The engineers concluded: “The failure was not due to any defective quality in the iron. It was not owing to the sudden effect of intense cold, for failure occurred by bending, and not by breaking. It was not the result of a weakness gradually developed after the erection of the bridge. It was due simply to the fact that it was not constructed in accordance with certain well established engineering principles” (Report 1877).

The engineer Albert H. Howland, perhaps on behalf of the coroner’s jury in Ashtabula, also reported to the joint committee on February 12, 1877. Howland estimated that each truss carried a dead load of 23.4 kN/m (1600 lb/ft) and an equivalent live load from the train of 20.4 kN/m (1,400 lb/ft) plus two concentrated vertical forces of 102.4 kN/m (23,000 lb). Such load values are largely in agreement with those estimated by Bowen, Johnson, and Graham; however, Howland widely overestimated wind pressures. Howland also performed static analyses for the gravity loads to determine the forces in the elements. Without explanation, he estimated “safe stresses” of 6.9 MPa (1,000 lb/sq in.) to 20.7 MPa (3,000 lb/sq in.) so that many of the compressive members had factors of safety less than one. His criticisms were the same as those of Bowen, Johnson, and Graham; further, he explicitly stated that some braces had moved from 1.3 to 3.8 cm (0.5 to 1.5 in). John D. Crohore, another engineer, disputed this: “But upon learning that the bridge was last painted two years ago, I re-examined the brace impressions, and found none that positively indicated any movement of the brace since the last painting and if they had not moved in the last two years, they were probably stable in the position they had assumed. I would not, of course, state positively that no braces were further displaced at the instant the fatal train struck the bridge but such an unusual displacement seems improbable” (Report 1877). Howland concluded that: “the failure began in the top chord of the south truss . . . at the second set of vertical rods counting from west to east; that it probably resulted from the buckling outward of the three I beams which are continuous at that point” (Report 1877).

A third major investigation of the failure was that of Charles MacDonald, probably done on behalf of the ASCE. His report was read before the ASCE on February 21, 1877. In agreement with other investigators,
MacDonald estimated that each truss carried a dead load of 20.4 kN/m (1,400 lb/ft) and that the equivalent live load from the train was 29.2 kN/m (2,000 lb/ft). He also calculated the member forces from the gravity loads, but did not calculate factors of safety; rather, he wrote in a very candid way about estimating the capacity of compressive members (MacDonald 1877).

But in passing to the compression members a serious difficulty is encountered in the attempt to determine a factor of safety from the strain per square inch, owing to the uncertain character of the data to be used in Gordon's formula, for the strength of columns. Let it be assumed that for columns having square end bearings, this expression would be for American iron; breaking weight per square inch = 40,000/(1 + (length)^2/40,000 x (rad. of gyration)^2). And for round ends, one-third of this amount. The value of radius of gyration squared for a beam of 9.6 inches section is 0.835. Substituting this in the expression and taking length for braces 260 inches, (inasmuch as the connection at center is very imperfect) we should have 13,300 pounds per square inch as the breaking resistance of one 6-inch beam having square end bearings and a length of 21 feet 8 inches. If it be assumed, that owing to imperfect end bearings, these braces had virtually round ends, the breaking resistance, 4,433 pounds would be below all but the two center sets in the bridge. The structure never would have even carried its own weight for a single day. But the facts disprove this assumption, and, unfortunately, we have nothing to guide us in deciding upon any figure between these limits.

Of even greater importance, MacDonald was the only engineer to note the existence of a flaw: "The cast iron angle block at top of second set of braces had the south lug broken off close to the face, and the line of fracture disclosed an air hole extending over one half the entire section." MacDonald concluded that "the failure first began in the south truss, at the second panel point from the west abutment," specifically at "the second top chord angle block" whose south lug was so "impaired by an air hole as to be reduced in strength fully one half" (MacDonald 1877) [See Fig. 3(c)].

**Assessment**

**Conceptual Design**

The dead load of the truss was not underestimated and the design live load of 36.5 kN/m (2500 lb/ft) was safely conservative. The Howe form does not longer-length compressive elements (the diagonals) and hence it is not the most economical system but it is certainly not inherently "excessively heavy." Moreover, in wood construction, tensile connections for the diagonals were not required. The height-to-span ratio of the bridge, a quantity that controls deflection, was set at a reasonable value of 7.8. The rather narrow 3.35-m (11-ft) panel length increased the flexural strength of the top chords and decreased the length of the diagonals. Having the top chords continuous over two panels also increased their flexural strength. The vertical lugs on the angle blocks at the second panel point on the top chord did have to transfer the axial forces from the I beams into the castings.

**Detailed Design**

The axial forces and stresses for the dead and live load condition were correctly computed by MacDonald and are shown in Fig. 2(b). MacDonald determined that the maximum axial stresses were as follows: in the top chord, 60 MPa (8,700 lb/sq in.); in the bottom chord 52.4 MPa (7,600 lb/sq in.); in the verticals, 54.4 MPa (7,900 lb/sq in.) and in the diagonal compressive braces, 35.9 MPa (5,200 lb/sq in.). Note that if the latter stress is multiplied by 6/4 to estimate the axial stress on four (the original number) rather than on six I beams, the value becomes 53.7 MPa (7,800 lb/sq in.). Therefore Tomlinson correctly computed the axial forces in the members from the dead and live load condition and he followed Stone's instruction to use an allowable stress of 55.2 MPa (8,000 lb/sq in.) for both tensile and compressive elements.

An allowable axial stress of 55.2 MPa (8,000 lb/sq in.) for wrought iron in tension is conservative; for slender compression elements the allowable stress should be a function of the slenderness of the elements and the rotational and translational restraints along the length and at the ends of the elements. By adding two I beams to the end braces, Stone in effect reduced the maximum axial stress in the braces to 35.9 MPa (5,200 lb/sq in.), a magnitude that apparently occurred many times on the Ashbula Bridge without incident. Adding the two I beams to the end braces was a change that even Tomlinson thought would have made the design acceptable to him (Report 1877):

**Q:** What I want to get at is, what was the main point of difference between you and Mr. Stone, in regard to the construction of the bridge?

**A:** As far as my memory goes, I never had any anxiety, except the compressive members of this bridge. I know that the chord rods were as good as could be. I don't think there was a better set of chords put into a bridge.

**Q:** I understand from that, that you had no fears, as far as the bridge was concerned, upon which the strain was tensile. And all your fears, and your anxiety, was about those parts which were in compression. What portion, or what part of the members were under compression, which gave you the greatest anxiety?

**A:** The three first set of braces in the bridge from the ends.

**Q:** By Mr. Pettibone: If, as you understand, that bridge was modified and the position of the braces changed, and the addition of other braces in the three end sets, would that in your judgment, have rendered the bridge perfectly safe?

**A:** If they were equal to the beams of the largest section furnished, I think that it would have been safe.

Of course the stresses computed by MacDonald were not the maximum stresses in the members. The tightening of the verticals induced axial pre-stress in the members and significant bending stresses occurred in the top chord. Indeed the compressive braces and the top chords would have been stronger if the I beams were continuously interconnected. Indeed the safety of the top chords would have increased had they not been subject to bending from the floor beams. Indeed there would have been greater safety against movement of the compressive braces if they had been mechanically connected to the angle blocks. Indeed, the diagonals should have had better end bearing. Indeed, because of slight length differences, some I beams were
more stressed than others. Nonetheless, the bridge performed safety for over 10 years.

Fabrication

The fabrication was done well. Tomlinson was especially proud of the tension elements: "The quality of the iron for the tension bars was all good. It was worked under my eye, and I know it to be good" (Report 1877). Some of the rolled sections were somewhat oversized, but not enough to increase axial stresses significantly. In retrospect, the transition from the lug to the main body of the angle block, where the solidification rates of the molten iron were so different, was a likely site for the formation of cracks or voids.

Erection

Stone should have retained Tomlinson to supervise the erection. Thus a correct cambor wall would have been set in the false work and the compressive chords would have fit (the first time) between the lugs. Tomlinson may have controlled the prestressing more precisely (although it was probably the excessive prestressing that convinced Stone to add I beams to the diagonal braces). The removal of the lugs intended to restrain the diagonal braces and the "chipping" of the ends of the braces probably did decrease strength, but with the additional I beams, the strength of the braces was, overall, increased.

Service

The satisfactory service for over ten years indicates that the static strength of the bridge was adequate, even without the modifications suggested by Bowen, Johnson, and Graham.

Investigations

Bowen, Johnson, and Graham, and MacDonald agreed in general on the magnitudes of the dead and live loads carried by the truss at the time of the collapse. They correctly computed the axial forces in the various members due to those loads. No estimates were made for the forces due to the tightening of the vertical rods nor for the flexure stresses in the top chord.

Neither MacDonald nor Bowen, Johnson, and Graham were able to show that the forces present in the compressive members at the time of failure exceeded their estimated capacities (Howland's strength estimates were mere guesses). Given that the load was ordinary, it cannot be said that failure occurred because of an one-time exceedance of a static strength of a member.

MacDonald correctly identified the cause of the collapse. A fatigue crack originated at the flaw in the lug and propagated under repeated stress cycles over 10 years. The fracture toughness of the iron was reduced by the very low temperatures on December 29, 1876 (for nearby Erie, Pa., the high temperature on that day was -3°C and the low temperature was -9°C). With the stress from the train, the crack [see Fig. 3(c)] became unstable and a brittle fracture occurred in the lug, causing the bridge to collapse. It is very likely that, if the flaw had not been there, the bridge would have served a full life.

Judgments and Consequences

The Joint Committee concluded that "the bridge went down under an ordinary load by reason of defects in its original construction," defects that "could have been discovered at any time after its erection by careful and analytical inspection." As defects, the Joint Committee listed the criticisms voiced by Bowen, Johnson, and Graham. The verdict of the coroner's jury at Ashtabula contained in essence the same points. In addition, the jury stated that: "Iron bridges were then in their infancy, and this one was an experiment which ought never to have been tried or trusted to span so broad and deep a chasm"; further, "that the responsibility of this fearful disaster and its consequent loss of life rests upon the Railway Company which by its Chief Executive Officer planned and erected this bridge" (Ashtabula News Extra, March 8, 1877). Neither the joint committee, nor the coroner's jury nor any of their engineers ever mentioned the flaw in the angle block casting. Such judgments aggravated the tragic human consequences of the failure. In addition to the persons who died in the collapse, Charles Collins "died by his own hand" after testifying before the joint committee. The Ashtabula Sentinel wrote that "Mr. Collins was about 51 years of age, of quiet, unassuming deportment" (Ashtabula Sentinel, January 25, 1877). Amasa Stone was, in effect, blamed for the tragedy. It is likely that the collapse was a reason for his own suicide, six years later, on May 10, 1883 (Johnson 1989). The judgment of Charles MacDonald was quite different (MacDonald 1877).

Twelve years ago, what was the extent of knowledge possessed by engineers on the subject of wrought-iron bridge building, judged by the work done, rather than what might have been derived from books? On the New England roads there were practically no iron bridges; that great trunk line, the Boston & Albany, still revelled in the security afforded by the 'principle of the Howe truss,' in wood; the New York Central, under the guidance of a foreign engineer, was experimenting in riveted work, now so much written against and used; none of the roads centering in New York had substituted iron for wood. The Pennsylvania Railroad, almost alone in that State, was but in the infancy of the effort which has since resulted in securing to her use some of the finest specimens of bridge architecture in the world. In the West a few scattering efforts had been made, and the subject was beginning to attract the attention of some of the best minds in the country. Squire Whipple, Albert Fink, Shaler Smith, Jacob H. Linville, and Thomas C. Clarke had built bridges at that time, it is true, but such names could almost be counted upon the fingers; and even these would, perhaps, now admit that they then 'built better then they knew.' If then, the state of knowledge at the time has not been under-estimated, the Ashtabula bridge was the result of an honest effort to improve the bridge practice of the country, undertaken by a man whose experience in wooden bridges warranted him in making the attempt.

MacDonald called for inspection of bridges by professionals and studies on the behavior of compression members. He stressed that: "care should always be taken not to pass abruptly from a large to a small mass, else the strains from cooling will surely vitiate the strength of the connection." By 1888 Copper's Specifications for bridge design explicitly forbade use of cast iron on any part of a bridge structure.

A broader consequence of the collapse was that the consulting engineers'
sieg on the master builder/entrepreneurial tradition of bridge building was
tightened (Jackson 1977). At one extreme was the sentiment of Edward S.
Philbrick who in a discussion to MacDonald's report stated: "I hope the
disaster may serve to teach railroad managers not to attempt too much
engineering themselves." Most of the investigating engineers were less blunt
but only one, Charles MacDonald, defended the master builder, Amasa
Stone.

The question of whether the entrepreneurial tradition of bridge building
led to an exceptional number of shoddily built, unsafe bridges remains to
be resolved. But at Ashtabula there’s no doubt that the builders were tech-
nically competent and wanted to achieve a “first class” (Report 1877),
innovative bridge using the best materials and workmanship available. It is
true, however, that the reliability of their conceptual design depended on
the reliability of a lug on an iron casting.

The investigating engineers, on their part, while flaunting their ability to
compute forces in statically determinate trusses, did not even mention the
effects of prestressing. They disagreed markedly on how to estimate the
strength of slender compressive elements and did not even allow the pos-
sibility of fatigue in iron.

**EPilogue**

The Joint Committee that investigated the Ashtabula collapse drafted
some provisions intended to become a bill before the Ohio Legislature. The
provisions were in essence a bridge design code, prescribing design loads,
allowable stresses, and minimum strengths, and requiring expert review of
design, construction supervision, and periodic inspections by engineers. It
was proposed that the expert reviewer “shall pass a successful examination
... before a committee of three members of the American Society of Civil
Engineers” (Report 1877). Even the joint committee admitted that “parties
will claim that the legislation is in the interest of engineers” (Report 1877).
The provisions did not become law. On receipt of the report, the Ohio
Legislature passed no additional resolutions on the collapse. Perhaps this
was due to Stone’s influence, but, nonetheless, it was a proper course for
a failure that was no one’s fault.

**Appendix. References**

*Ashtabula sentinel.* (1877). Jan. 11, 5.


Cooper, T. (1888). *General specifications for iron and steel railroad bridges and


*Timeline,* Jun.-Jul.

Society of Civil Engineers,* Feb., 74–87.

Mahan, D. H. (1837). *An elementary course in civil engineering.* Wiley and Putnam,
New York, N.Y.

*Report of the joint committee concerning the Ashtabula Bridge disaster.* (1877). Nevins
and Myers State Printers, Columbus, Ohio.

Collapse.” *Timeline,* Jun.-Jul.

8.

N.Y.