Forbes Avenue Over Fern Hollow Bridge Collapse Investigation:

Assessment of Bridge Inspection and Load Rating

Prepared For:

National Transportation Safety Board

NTSB Accident ID: HWY22MH003

Prepared jointly by:

Derek Soden, P.E., S.E.
Federal Highway Administration
Office of Bridges and Structures
1200 New Jersey Avenue, SE
Washington, DC 20590

Lubin Gao, Ph. D., P.E.
Federal Highway Administration
Office of Bridges and Structures
1200 New Jersey Avenue, SE
Washington, DC 20590

Samantha Lubkin, P.E.
Federal Highway Administration
Office of Bridges and Structures
1200 New Jersey Avenue, SE
Washington, DC 20590

Dennis O'Shea, P.E.
Federal Highway Administration
Office of Bridges and Structures
1200 New Jersey Avenue, SE
Washington, DC 20590

Jonathan Buck, P.E.
Federal Highway Administration
Pennsylvania Division Office
30 North Third Street, Ste 700
Harrisburg, PA 17101

Justin Ocel, Ph. D., P.E.
Federal Highway Administration
Resource Center Structures Team
31 Hopkins Plaza, Suite 840
Baltimore, MD 21201

September 14, 2023

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREVIATIONS

Item	Definition
AASHTO	American Association of State and Highway Transportation Officials
ABET	Accreditation Board for Engineering and Technology
ASR	allowable stress rating
BIRM	Bridge Inspector's Reference Manual (FHWA term)
BITC	bridge inspection training catalog (PennDOT term)
CAD	computer-aided drawing
CBSI	certified bridge safety inspector (PennDOT term)
CFR	Code of Federal Regulations
CL	centerline
CVN	Charpy V-notch
DL	dead load
FB	floor beam
FCM	fracture critical member
FHWA	Federal Highway Administration
GVW	gross vehicle weight
LL	live load
LFR	load factor rating
LRFR	load and resistance factor rating
MBE	Manual for Bridge Evaluation (AASHTO term)
MBEI	Manual for Bridge Element Inspection (AASHTO term)
MTR	mill test report
NBI	National Bridge Inventory
NBIS	Nation Bridge Inspection Standards
NDT	nondestructive testing
NHS	National Highway System
NICET	National Institute for Certification in Engineering Technologies
NTE	not to exceed
NTSB	National Transportation Safety Board
PE	professional engineer
PennDOT	Pennsylvania Department of Transportation
RF	rating factor
SDOT	state department of transportation
SLC	safe load capacity (PennDOT term)
UWS	uncoated weathering steel

INTRODUCTION

FEDERAL HIGHWAY ADMINISTRATION (FHWA) ROLE

The Fern Hollow Bridge collapse investigation was led by the National Transportation Safety Board (NTSB). The Federal Highway Administration (FHWA) supported NTSB in its investigation by providing resources and expertise to evaluate the bridge design, bridge materials, past bridge inspection reports, and load rating reports. The FHWA team consisted of personnel from the Office of Bridges and Structures, the Pennsylvania Division Office, the Resource Center, and the Turner-Fairbank Highway Research Center.

REPORT PURPOSE

This report focuses on the findings from FHWA's review of past bridge inspections conducted between 2005 and 2021 and a verification of prior load ratings. The report focuses on the requirements for bridge inspection and load rating, adherence to relevant laws and regulations, and gaps identified in the inspection and evaluation process.

BRIDGE DESCRIPTION

The Fern Hollow Bridge carried Forbes Avenue over Fern Hollow and 9 Mile Run through Frick Park within the City of Pittsburgh, Pennsylvania. The bridge used a rigid, K-frame superstructure type as shown in the elevation view within Figure 1. Being a rigid frame implies that the main longitudinal girders act integrally with the intermediate legs supporting the middle of the bridge. With the incline of the legs, the rigid frame structure type is further defined as a "K-frame", as the entire superstructure looks similar to the letter "K" turned on its side. The K-frames were fabricated members, built up from steel plates welded together to form I-shaped sections of varying depth. Each end of the bridge was supported on abutments, and the legs rested upon foundation elements called piers, which acted as thrust blocks resisting axial loads and shear in the legs.

A plan view of the bridge's floor framing system is shown in Figure 1 and a half cross-section is shown in Figure 2. Loads from the bridge deck were distributed into seven equally spaced rolled wide flange stringers that rested upon deeper floor beams that, in turn, distributed loads to the two exterior K-frame girders. Figures 1 through 5 highlight nomenclature used to describe and identify elements of the bridge throughout this report. The nomenclature was adopted from the inspection reports and was derived from the perspective of a person standing at the west abutment and facing east. Thus, the west abutment was the "near" abutment, and the east abutment was the "far" abutment. Spans, bents, and floor beams were all labelled with sequential numbers going from the near to far abutment. The left- and right-hand sides of the bridge are taken from the perspective of a person standing at the near end facing the far end. In relation to cardinal directions, east and far were synonymous, as were west and near, north and left, and south and right. Lastly, when the terms "inside" and "outside" were used in reference to a bridge element, "inside" is the face towards the center of the bridge, when underneath the deck, and "outside" is the face away from the center of the bridge.

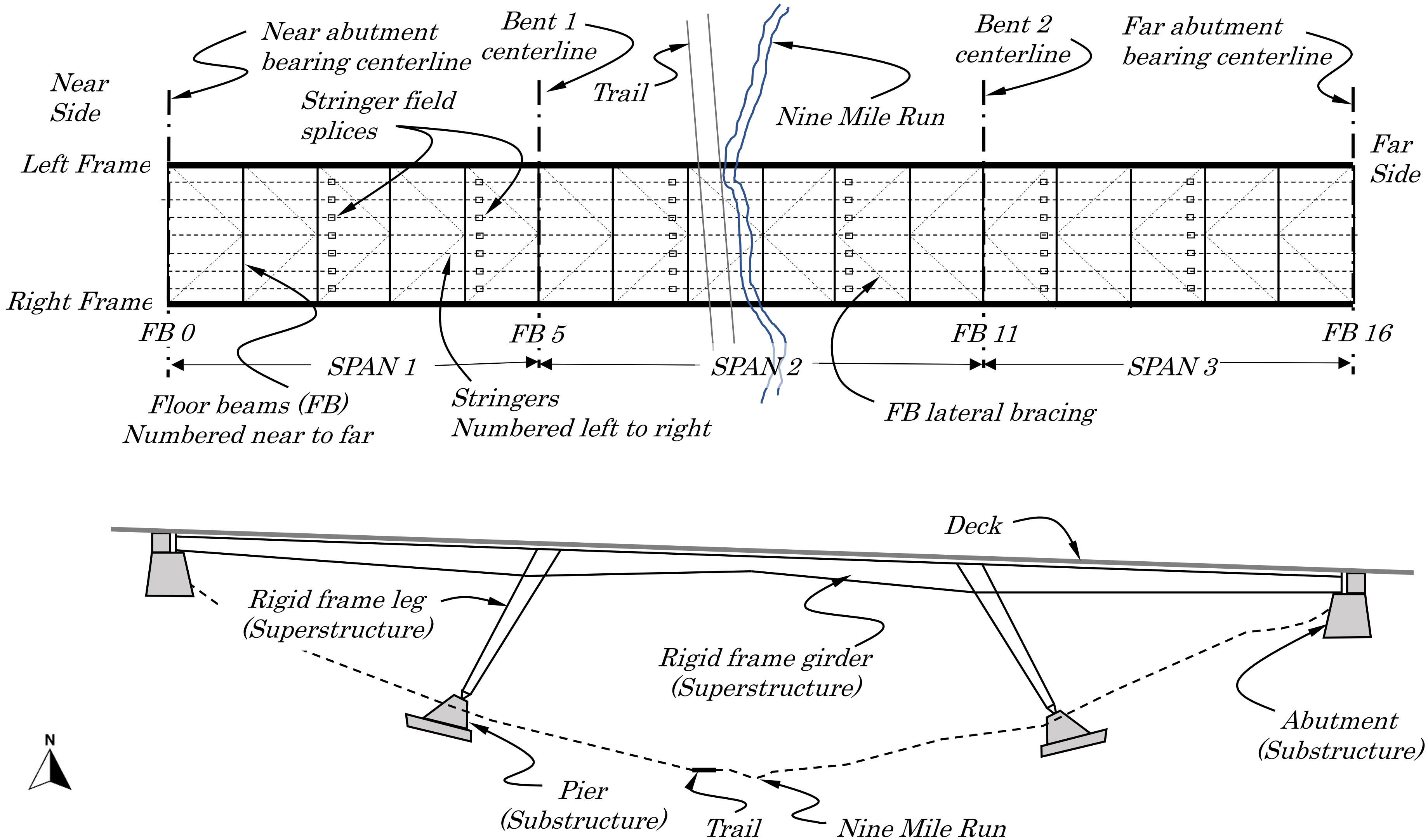


Figure 1. Plan view/framing plan and elevation of Fern Hollow Bridge showing relevant terminology and orientation.

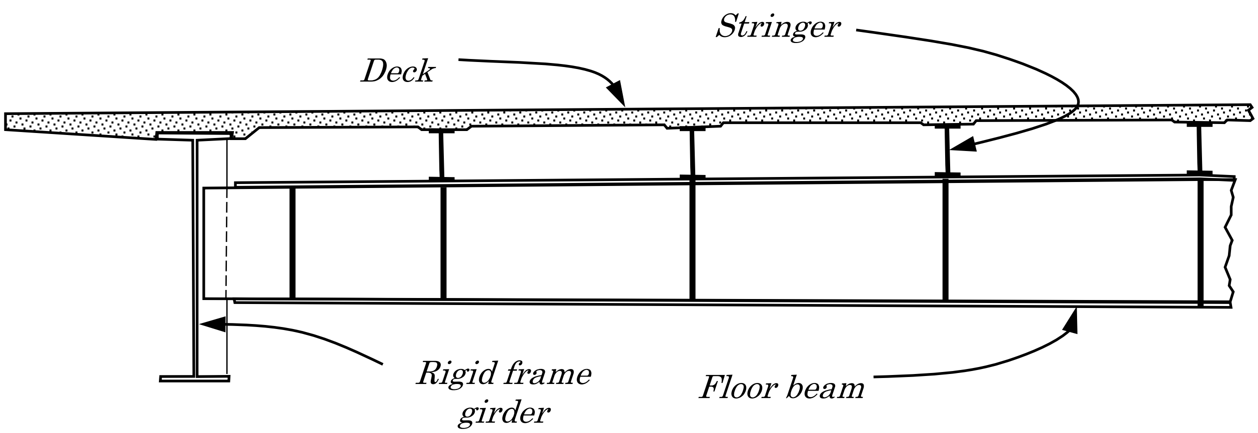


Figure 2. Half section of the Fern Hollow Bridge showing relevant terminology.

Where used in design and inspection documents, the term “bent” referred to the laterally stiffened assembly of right and left frame legs as shown in Figure 3. The right and left legs at each bent were originally connected by two cross braces (referred to as “X-braces” or “diagonal braces” in some documents) fabricated from square steel tube sections. With time, advancing deterioration of the cross frames led to concerns about their capability to brace the bridge, which led the bridge’s owner to supplement the braces with steel cables arranged in an “X” arrangement on the near and far faces of both bents (shown in Figure 3), primarily to resist wind loading. In 2018, the lower cross brace at Bent 1 was removed, ostensibly to reduce the danger of it falling onto the trail below.

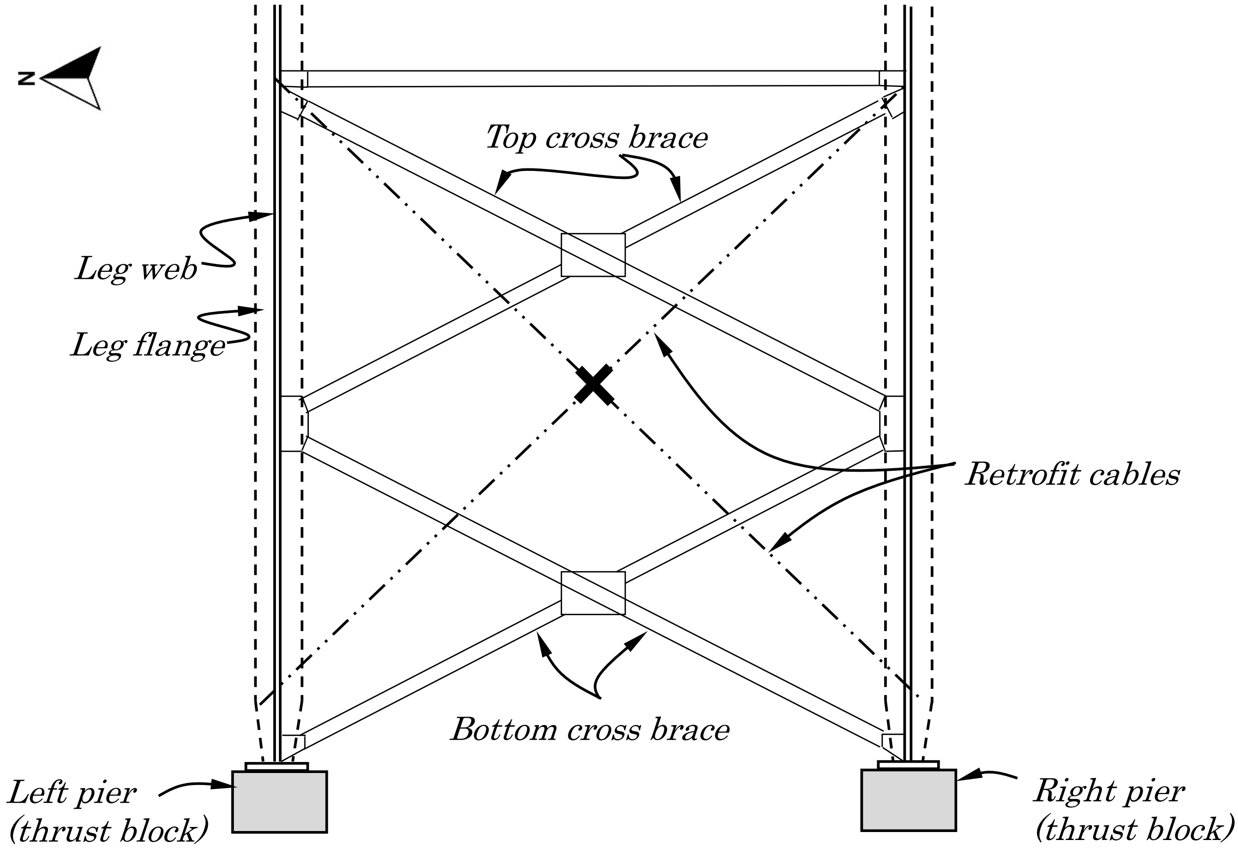


Figure 3. Near face of Fern Hollow Bridge Bent 2 showing relevant terminology.

The girders and legs of the rigid K-frame were built-up I-shapes (i.e. two flanges separated by a web) as shown in Figure 4. The entire frame was optimized for weight, thus there were many transitions in plate thickness for both flanges and webs along the length of the members. The web plates in both the girders and legs were relatively thin, requiring both longitudinal and transverse stiffeners to suppress local buckling modes. Although an adequate design, this frame could be considered atypical in that the legs were attached to the main girders via a bolted endplate connection, whereas most K-frames detail this connection with a fully welded integral web plate.

The frame legs were inclined at a 30-degree angle, with respect to the vertical axis of the frame girder, and tapered in depth, from approximately 6’-10½ ” deep at the connection to the girder to just over 3’-7” at a point 3’-2” above the base of the leg. At that point, the tapering angle steepened toward the thrust block, forming a region referred to as the “shoe” in design documents and shown in Figure 4. The shoe region was capped with an eight-inch-deep steel “toe” recessed in a 7” thick masonry plate that beared on the thrust block, providing axial and shear resistance while not restraining rotations. This region of the leg is referred to as the “shoe” in Figure 4. The change in direction of the leg flange plate at the top of the shoe introduced tension in the leg perpendicular to its longitudinal axis. A plate at the top of the shoe resisted this tension. This plate is labeled a transverse stiffener on the original construction plans for the bridge, but it was much larger than other stiffeners in the leg due to its additional function.

Given the role that this element caused in the collapse, it will be called the shoe tension tie throughout this report. Details of the shoe are shown in Figure 5.

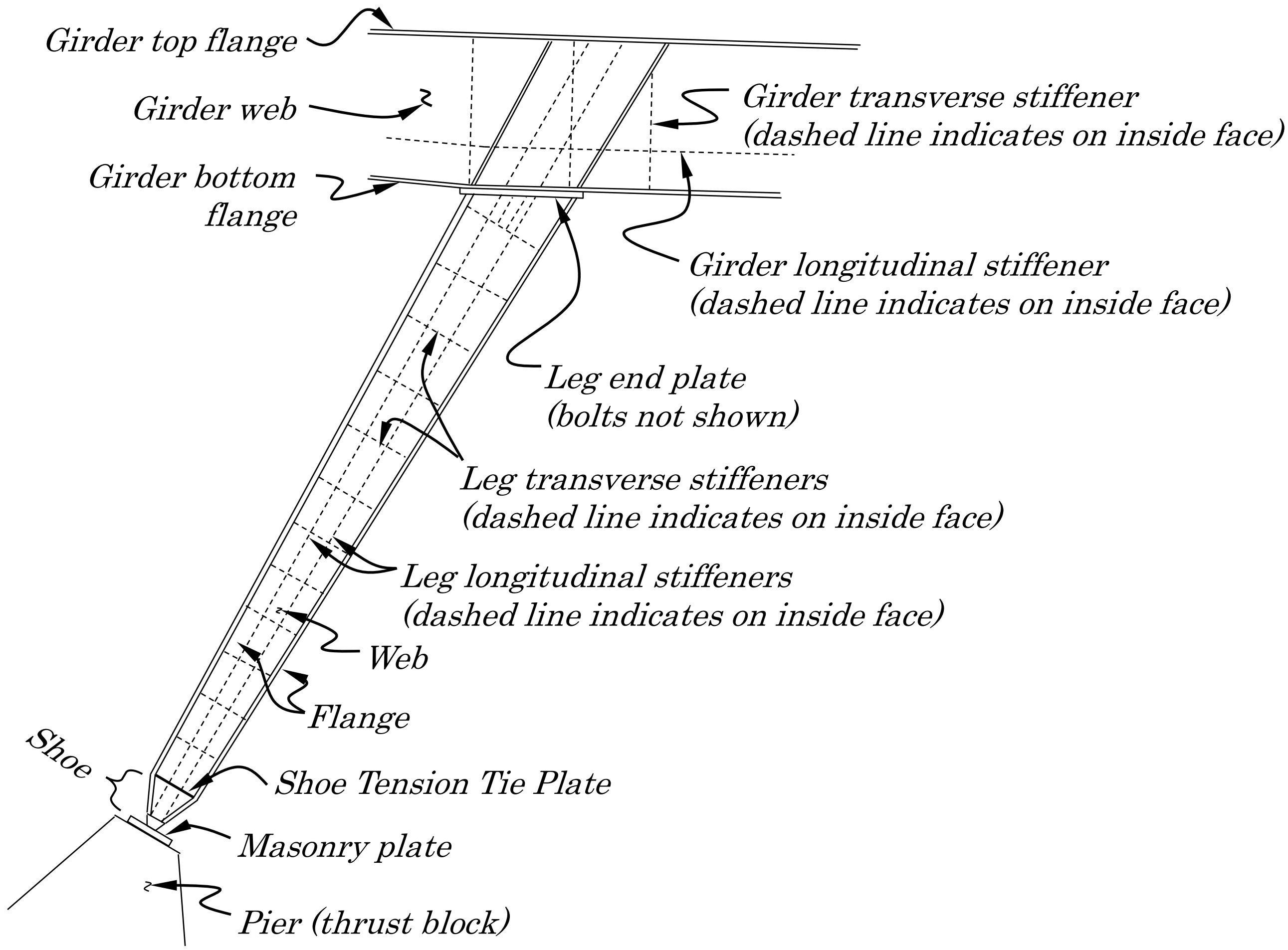


Figure 4. Fern Hollow Bridge frame leg elevation view (shown from outside of bridge) showing relevant terminology.

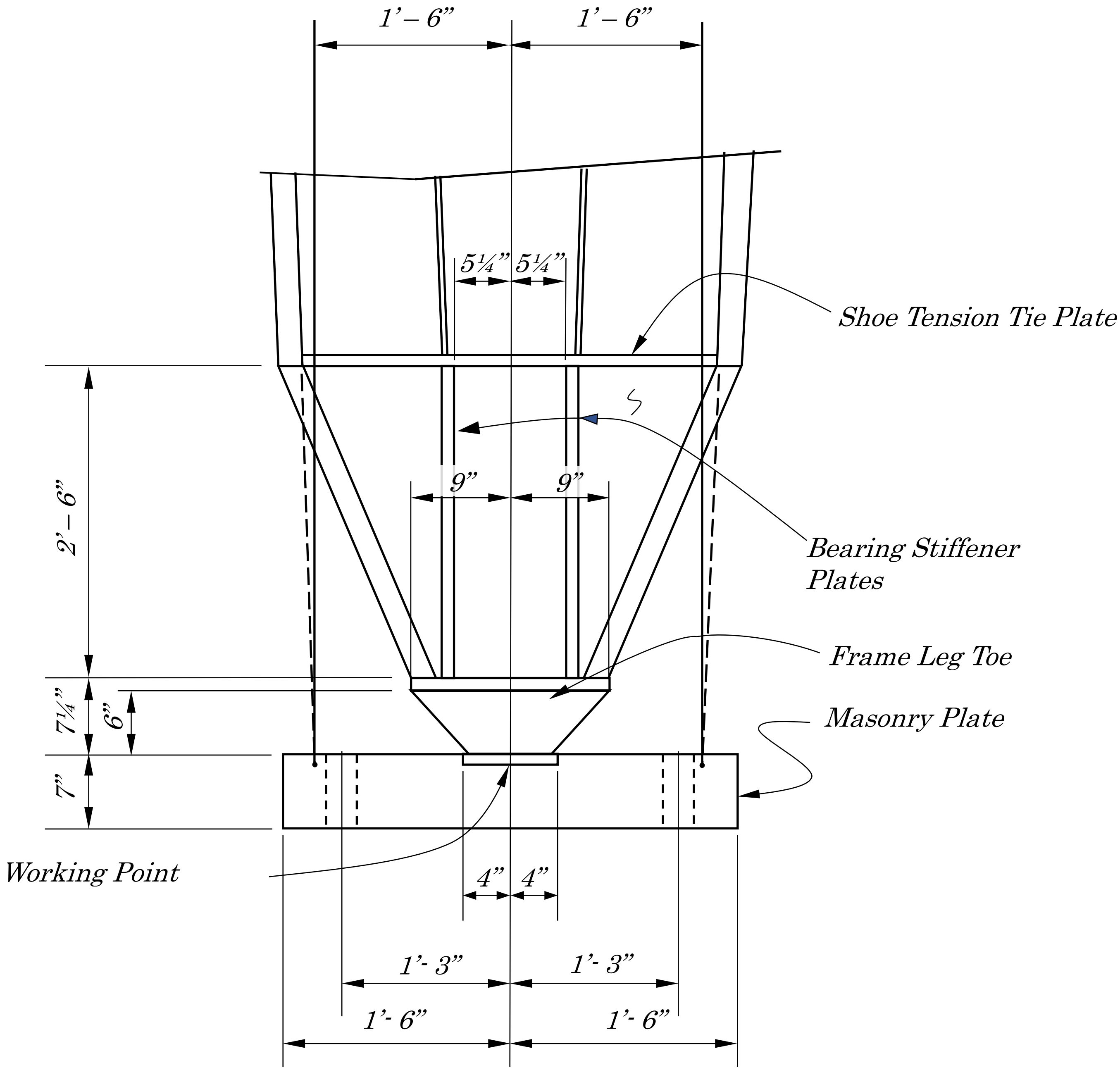


Figure 5: Details of the rigid frame leg shoe

The bridge design was completed by Richardson, Gordon, and Associates of Pittsburgh, PA in late 1970. The bridge owner was unable to supply shop drawings for the bridge after the collapse but mill test reports (MTRs) for the steel were retained and provided. The MTRs show the bridge was fabricated by Conn Construction Company of New Castle, PA. Based on the dates of the MTRs, the superstructure was fabricated between March and August 1972 and, based on the purchase of bolts and bearing pads, it was likely erected in late 1972 to early 1973. All the steel for the superstructure was specified to be ASTM A588 and was delivered to both Grades A and B with a supplementary Charpy V-notch (CVN) requirement of 15 ft-lbf at 40°F. Steel for the bridge was sourced from both Bethlehem Steel Corporation and the United States Steel

Corporation. ASTM A588 is referred to as an uncoated weathering steel (UWS) and is intentionally not painted. For corrosion protection, small amounts of copper, nickel, and chrome in UWS develop a stable rust patina, resulting in a natural brown color appearance. When installed in a suitable location, and properly detailed and maintained, UWS is a considered an economical alternative to painting, as it does not require periodic recoating.

COLLAPSE OVERVIEW

The bridge collapsed suddenly in the early morning of January 28, 2022. A plan view of the collapse site taken by a drone is shown in Figure 6 and is in the same cardinal orientation as Figure 1. There were four passenger vehicles and one commuter bus on the bridge at the time of the collapse. The temperature at that time was approximately 26°F, with no wind and light snow. As seen in Figure 6, the east end of the bridge is displaced much further from its abutment than the west side of the bridge. This arrangement indicated that the collapse sequence started on the west end of the bridge with the west end of the bridge hitting the ground first and the middle and east spans collapsing toward the middle of the bridge and Nine Mile Run, dragging the east span down the slope. Video evidence captured from the commuter bus crossing the bridge during the collapse confirmed this sequence.

Figure 6. Drone image of collapse site.

Investigators on-site shortly after the collapse did not find any primary fractures in the floor system or girders of the K-frames. Evidence indicated the frame legs as the initiators of the collapse sequence, in particular the Bent 1, right (i.e. southwest) leg. Figures 7 through 10 are photos of the bottom of the frame legs taken after other bridge components had been removed. As seen in those figures, the bottom of the two left legs (Figures 7 and 8) were mostly intact, but the two right legs (Figures 9 and 10) were more distressed. The Bent 1 right leg was kinked in the I-shape's strong axis at the shoe with the far-face flange completely missing. The Bent 2 right leg shoe was intact, but the first web panel over the shoe had fractured along one of the web-to-flange welds and one flange had buckled into the web. It was also observed that the loss of section due to corrosion in the web plates of all four legs was severe to the point where there were holes in numerous locations.

The distress observed in the Bent 1 frame legs, in addition to the commuter bus video, led FHWA investigators to consider the Bent 1, right, shoe as being principally involved in the collapse sequence. Thus, FHWA's evaluation of bridge inspection and load rating records for the Fern Hollow Bridge focused on the treatment of the frame legs in bridge inspection and load rating.

Figure 7. Bent 1 left leg.

Figure 8. Bent 2 left leg.

Figure 9. Bent 1 right leg.

Figure 10. Bent 2 right leg.

REVIEW OF FERN HOLLOW BRIDGE INSPECTIONS

The City of Pittsburgh provided bridge inspection reports from 2005 to 2021. FHWA reviewed these reports to assess adherence to inspection regulations, standards, and accepted practices. Comparing the reports across years also provided insights into the progression of deterioration observed on the bridge and how the bridge was being managed by the City of Pittsburgh with respect to maintenance recommendations resulting from bridge inspections.

GOVERNING REGULATIONS AND STANDARDS FOR BRIDGE INSPECTION

The National Bridge Inspection Standards (NBIS) are the regulations governing safety inspection and evaluation of all public highway bridges in the United States in accordance with 23 U.S.C. 144. Congress required the Department of Transportation to establish these standards in 1968 following the 1967 collapse of the Silver Bridge over the Ohio River between West Virginia and Ohio. The NBIS was first published in 1971 (36 FR 7851), creating the Nation's first nationally coordinated bridge inspection program. Updates to the standards have been made since 1971, including a recently published 2022 revision[1].

Periodic and thorough inspections of bridges are necessary to maintain safe bridge operation, prevent structural and functional failures, and to provide data on the condition and operation of bridges that is necessary for owners to make informed investment decisions as part of an overall asset management program.

The NBIS is codified in 23 CFR 650 Subpart C. At the time of the collapse, 23 CFR 650.317 incorporated by reference the American Association of State Highway and Transportation Officials (AASHTO) *Manual for Bridge Evaluation* (MBE) First Edition, 2008[2]. The MBE includes standards for inspection procedures, reporting, and techniques, and load ratings.

At the time of the collapse, 23 CFR 650.315 required data collection in accordance with the FHWA's December 1995 *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* (otherwise known as the Coding Guide). The Coding Guide contains descriptions of the inventory data items that are to be reported annually by the states to the FHWA National Bridge Inventory (NBI) for all highway bridges in their jurisdiction, including locally owned highway bridges like the Fern Hollow Bridge. The Coding Guide includes inventory data items (information that does not change or changes infrequently such as latitude/longitude or structure type), as well as inspection data (such as summary component condition ratings) and appraisal data ratings that are updated routinely through the life of the bridge.

The FHWA additionally publishes the Bridge Inspector's Reference Manual (BIRM). The BIRM covers all aspects of bridge inspection, from planning to performance, and presents

[1] Between the time of the collapse and when this report was submitted to NTSB, the National Bridge Inspection Standards were revised and finalized in May 2022. This report's findings are based on the 2004 version of the NBIS (69 FR 74419) with 2009 revisions (74 FR 68379), which was in effect at the time of the bridge collapse.

[2] The 2022 revisions to the NBIS incorporated the MBE 3rd Edition, 2018, including 2019 and 2020 interim revisions. References to the MBE in this report will be based on the 1st Edition as it was the incorporated version at the time of the collapse.

inspection techniques to be used in specific locations by structure type and structure material. The BIRM is also the basis for the National Highway Institute's (NHI) comprehensive bridge inspection training courses (NHI-130055 and NHI-130056).

Key Regulatory Requirements for Inspections

The NBIS in effect at the time of the collapse was published in December 2004, with a revision in 2009 to update incorporated references. This version of the NBIS was the regulation active at the time of all inspections of the Fern Hollow Bridge reviewed for this investigation. Relevant sections of this regulation are as follows:

23 CFR 650.305 – Definitions.

Pertinent definitions for this report include:

- Fracture critical member (FCM). "A steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse."
- Fracture critical member inspection. "A hands-on inspection of a fracture critical member or member components that may include visual and other nondestructive evaluation."
- Hands-on. "Inspection within arms-length of the component. Inspection uses visual techniques that may be supplemented by nondestructive testing."
- In-depth inspection. "A close-up, inspection of one or more members above or below the water level to identify any deficiencies not readily detectable using routine inspection procedures; hands-on inspection may be necessary at some locations."
- Program manager. "The individual in charge of the program, that has been assigned or delegated the duties and responsibilities for bridge inspection, reporting, and inventory. The program manager provides overall leadership and is available to inspection team leaders to provide guidance."
- Routine inspection. "Regularly scheduled inspection consisting of observations and/or measurements needed to determine the physical and functional condition of the bridge, to identify any changes from initial or previously recorded conditions, and to ensure that the structure continues to satisfy present service requirements."
- Special inspection. "An inspection scheduled at the discretion of the bridge owner, used to monitor a particular known or suspected deficiency."
- Team leader. "Individual in charge of an inspection team responsible for planning, preparing, and performing field inspection of the bridge."
- Critical finding. "A structural or safety related issue that requires immediate follow up action."

23 CFR 650.307- Bridge inspection organization

Requirements from 23 CFR 650.307 pertinent for this report include:

- Each State transportation department must include an inspection organization to carry out all NBIS requirements and inspect, or cause to be inspected, all highway bridges on public roads that are fully or partially located within the State's borders[3].
- This organization is required to have a Program Manager who is responsible for inspections, reporting, and inventorying, and provides overall leadership and guidance to inspectors and Team Leaders[4].
- Inspection organization functions may be delegated (e.g., to local agencies), but delegation does not relieve the State transportation department of its NBIS responsibilities[5].

23 CFR 650.309- Qualifications of Personnel

This section details requirements for the qualifications of:

- Program managers. Program managers must have 10 years bridge inspection experience, or be a register professional engineer, and have completed an approved comprehensive bridge inspection training course[6].
- Team leaders. There are five ways to qualify as a team leader for inspections through a combination of experience and training:
    1. Meet the qualifications for a program manager[7].
    2. Have 5 years of bridge inspection experience and have completed an approved comprehensive bridge inspection training course[8].
    3. Be a National Institute for Certification in Engineering Technologies (NICET) Level III or IV Bridge Safety Inspector and have completed an approved comprehensive bridge inspection training course[9].
    4. Have all the following[10]:
        - A Bachelor's Degree in engineering from an accredited or determined as substantially equivalent by the Accreditation Board for Engineering and Technology (ABET),
        - Passed the Fundamentals of Engineering exam,
        - Have 2 years of bridge inspection experience, and
        - Have completed an approved comprehensive bridge inspection training course.
    5. Have all the following[11]:
        - An Associate's Degree in engineering from an accredited or determined as substantially equivalent ABET,
        - Have 4 years of bridge inspection experience, and

[3] 23 CFR 650.307(a) and 23 CFR 650.307(c)
[4] 23 CFR 650.307(e)
[5] 23 CFR 650.307(d)
[6] 23 CFR 650.309(a)
[7] 23 CFR 650.309(b)(1)
[8] 23 CFR 650.309(b)(2)
[9] 23 CFR 650.309(b)(3)
[10] 23 CFR 650.309(b)(4)
[11] 23 CFR 650.309(b)(5)

    - Have completed an approved comprehensive bridge inspection training course.
- Load rating engineer. The individual with overall responsibility for load rating bridges. The Load rating engineer must be a professional engineer[12].

23 CFR 650.311- Inspection Frequency

Requirements from 23 CFR 650.311 pertinent for this report include:

- Routine inspections. Typically performed at an interval not to exceed (NTE) 24 months[13]. Certain bridges may, based on condition or other risk factors, require inspection at an interval less than 24 months, based on criteria developed by the State transportation department[14].
- Fracture critical member inspections. Typically performed at an interval NTE 24 months[15]. Risk factors may necessitate inspection at an interval less than 24-months[16].

23 CFR 650.313- Inspection Procedures

Requirements from 23 CFR 650.313 pertinent for this report include:

- All bridges are to be inspected in accordance with the AASHTO MBE[17].
- A qualified team leader must be present at all times during routine, in-depth, FCM, and underwater inspections[18].
- Each bridge must be load rated in accordance with the MBE and posted in accordance with the MBE or State law when the maximum unrestricted legal loads or State routine permit loads exceed that allowed by the operating, or equivalent, rating factor.[19]
- Bridge files must be prepared in accordance with the MBE, including documentation of actions taken to address inspection findings, relevant maintenance and inspection data, and recordation of findings on standard forms[20].
- Bridges with FCMs must include procedures which identify the location of the FCM, the interval of FCM inspection, and the FCMs must be inspected in accordance with those procedures[21].
- The inspection organization must include systematic quality control (QC) and quality assurance (QA) procedures to maintain high degree of accuracy and consistency. The procedures must include periodic field review of inspection teams, periodic inspection

[12] 23 CFR 650.309(c)
[13] 23 CFR 650.311(a)(1)
[14] 23 CFR 650.311(a)(2)
[15] 23 CFR 650.311(c)(1)
[16] 23 CFR 650.311(c)(2)
[17] 23 CFR 650.313(a)
[18] 23 CFR 650.313(b)
[19] 23 CFR 650.313(c)
[20] 23 CFR 650.313(d)
[21] 23 CFR 650.313(e)(1)

refresher training for PMs and TLs, and independent review of reports and computations[22].
- A policy must be established to assure critical findings are addressed in a timely manner, and the State transportation department must inform FHWA of the actions taken to resolve or monitor these findings[23].

Pertinent AASHTO MBE Requirements

The following are pertinent requirements from the AASHTO MBE 1st Edition for inspection procedures that apply to the observations made in the inspection reports reviewed for the Fern Hollow Bridge.

- MBE Article 4.8.1.2- Cleaning. "*Metal structures with heavy plate corrosion will require chipping with a hammer or other means to remove corrosion down to the base metal in order to measure the remaining section*."
- MBE Article 4.8.3.1- Steel beams, girders, and box sections. "*Structural steel members should be inspected for loss of section due to corrosion. Where a build-up of rust scale is present, a visual observation is usually not sufficient to evaluate section loss. Hand scrape areas of rust scale to base metal and measure the remaining section using calipers, ultrasonic thickness meters, or other appropriate method. Sufficient measurements should be taken to allow the evaluation of the effect of the losses on member capacity…*"
  "*Check for fatigue cracks which typically begin near weld terminations of stiffeners and gusset plates due to secondary stresses or out-of-plane bending. Any evidence of cracking should be carefully documented for evaluation and appropriate follow-up, as necessary…*"
  "*Inspect uncoated weathering steel structures for details or conditions that promote continuous wetting of uncoated steel; bridge geometrics that result in salt spray reaching the uncoated steel; pitting of the surface of the steel indicating unacceptable degradation of the steel*."

FHWA REVIEW OF FERN HOLLOW BRIDGE INSPECTIONS

FHWA's observations from their review of inspection reports for the Fern Hollow Bridge from the period between 2005 to 2021 are provided in subsequent sections:

Inspection Organization

With the bridge being located in the state of Pennsylvania, The Pennsylvania Department of Transportation (PennDOT) is responsible for bridge inspection program management. PennDOT delegated functions of the program to the City of Pittsburgh for bridges they own, although this delegation does not relieve PennDOT of overall responsibility.

[22] 23 CFR 650.313(g)
[23] 23 CFR 650.313(h)

Inspection Intervals and Timeliness

FHWA reviewed inspection reports to assess whether the Fern Hollow Bridge was inspected at the intervals required by the NBIS and PennDOT policy.

- Routine inspections were performed at 24-month intervals, the maximum allowed by the NBIS, in the period reviewed.
- FCM inspections were also performed at the maximum 24-month interval allowed by the NBIS, usually commensurate with routine inspections.
- After the 2013 routine inspection, the interval between FCM inspections was reduced to 12-months. This reduction was based on PennDOT policy (PennDOT Publication 238, Table IP 2.3.2.4-1), which states that once the superstructure general condition rating (as recorded in the NBI) drops to 4 (Poor) or lower, or has a posted load restriction, the interval between FCM inspection is to be reduced to 12-months and additional Other Special (OS) inspections are required. According to PennDOT Publication 238, the purpose of an OS inspection is to "*monitor posted bridges; poor, serious and critical condition ratings; bridges with severe scour issues or known high priority maintenance recommendations and fulfill the need for more frequent inspections*." From the inspection reports, the scope of the Other Special inspections included the floor beams, rigid frame girders and legs, and load posting signs and excluded the stringers, lateral bracing, deck, and concrete substructures.
- After the September 2017 routine inspection, the interval between FCM and OS inspections was further reduced to 6 months. This was triggered by PennDOT policy and due to an open or deferred Priority 0 or 1 maintenance item. In this case, the September 2017 inspection documented that the connection between the Bent 1 lower cross brace and the frame legs had corroded to the point where they were nearly severed, which the inspection report cited as a safety concern due to the potential for it to fall. Addressing this deterioration was assigned to be a Priority 1 maintenance item for the City of Pittsburgh.
    - A March 2018 FCM and OS inspection was performed with continued observation that the lower cross brace connection on Bent 1 was nearly severed and needed to be remedied.
    - The September 2018 FCM and OS inspection observed that the lower cross brace of Bent 1 was completely severed and resting against the steel cables installed in 2008 to supplement the cross frames. This was assigned a Priority 0 maintenance item for the City of Pittsburgh. The City of Pittsburgh removed the lower cross brace at Bent 1 before the next scheduled FCM and OS inspection scheduled for March 2019. As the priority maintenance item was resolved, the bridge returned to a 12-month interval for FCM and OS inspection.

Inspection Teams and Personnel Qualifications

FHWA reviewed inspection reports to confirm that qualified personnel were engaged in the inspections. Through this review, FHWA verified and confirmed experience and training qualifications. All team leaders were found to be qualified under the NBIS and PennDOT policy.

Fracture Critical Member Identification

Fracture critical member inspection is expected to identify defects that, if unaddressed, could lead to member failure and cause sudden partial or full collapse of the bridge. Fracture critical member inspection is required by the NBIS to be more rigorous than routine inspection, with the expectation that inspectors get within arms-reach of the member or member component being inspected ("hands-on") and, as deemed necessary, supplement visual and physical inspection with nondestructive testing.

The requirement for bridge owners to identify fracture critical members and develop and maintain inspection procedures for those members was first included in the NBIS in 1988. Fracture critical member inspection was established as an inspection type separate from routine inspection in the 2004 update to the NBIS (the version in effect at the time of the collapse). Over time, the term "fracture critical" came to be perceived as denoting a higher likelihood of failure, which is not the case. Thus, in the 2022 update to the NBIS, fracture critical member inspection was renamed nonredundant steel tension member (NSTM) inspection. The change in name is intended to refocus efforts on identifying members with a higher consequence associated with sudden failure (full or partial collapse), relative to other members, and implement inspection procedures that will identify defects that could result in localized failures.

To be considered a Fracture Critical Member, a bridge component must meet all three of the following criteria from the definition of FCM in 23 CFR 650.305:

- It is steel. This criterion can be assessed through observation.
- It is fully or partially in tension. In some structure types, his criterion can be assessed through observation with a rudimentary understanding of structural mechanics. However, in structures like the Fern Hollow Bridge, where a member can be subject to both axial compression and bending, structural analysis would be required to determine whether the tension component of the bending stress exceeds the compression due to axial load.
- Failure of the member (e.g., complete fracture) will cause the bridge to partially or fully collapse. This criterion is assessed through an evaluation of the level of redundancy in the structure. The FHWA course entitled "Fracture Critical Inspection Techniques for Steel Bridges" (NHI-130078) identifies three types of redundancy; load-path, structural, and internal. Prior to the 2022 revisions to the NBIS, FHWA recognized only load path redundancy as a means to avoid collapse in the event of a member failure, for the purpose of identifying FCMs.

  Load path redundancy is defined in the June 20, 2012, FHWA memo "Clarification of the Requirements for Fracture Critical Members" as being based on the number of main supporting members between points of support, usually parallel, such as girders or trusses[24]. Typical practice would be to consider a bridge with three or more parallel

[24] The 2012 memo was rescinded on May 9, 2022 and replaced with updated guidance more consistent with the 2022 NBIS Final Rule, but was in effect during the time of the collapse.

girders to possess load path redundancy, whereas a bridge with two parallel girders would be considered non-load path redundant.

FHWA reviewed inspection reports for the Fern Hollow Bridge to determine whether fracture critical member inspection was required.

- As the Fern Hollow Bridge was supported by only two parallel rigid frame lines, the frames lacked load path redundancy.
    - As flexural members with little axial load (aside from thrust from the frame legs), the rigid frame girders would be partially in, making them FCMs.
    - The rigid connection between the rigid frame girders and frame legs allowed for the transfer of bending moments in the frame girders to the top of the frame legs. This would add to moments resulting from the eccentricity of the vertical reaction from the frame girders to the reaction at the base of the inclined leg. Where these combined bending stresses exceeded the axial compression stress in the legs due to thrust, the frame legs would be partially in tension.
    - A "Frame Thrust & Moment Stress Table" included on sheet 12 of the original construction plans indicates that this was the case near the top of the legs. That table reports design axial stresses in the legs of 7.3 kips per square inch (ksi) and bending stresses of ±14.4 ksi. Structural analysis performed by FHWA after the collapse (Figure 11) confirmed these values and found that the top two-thirds of the frame leg would experience tension under design loads.

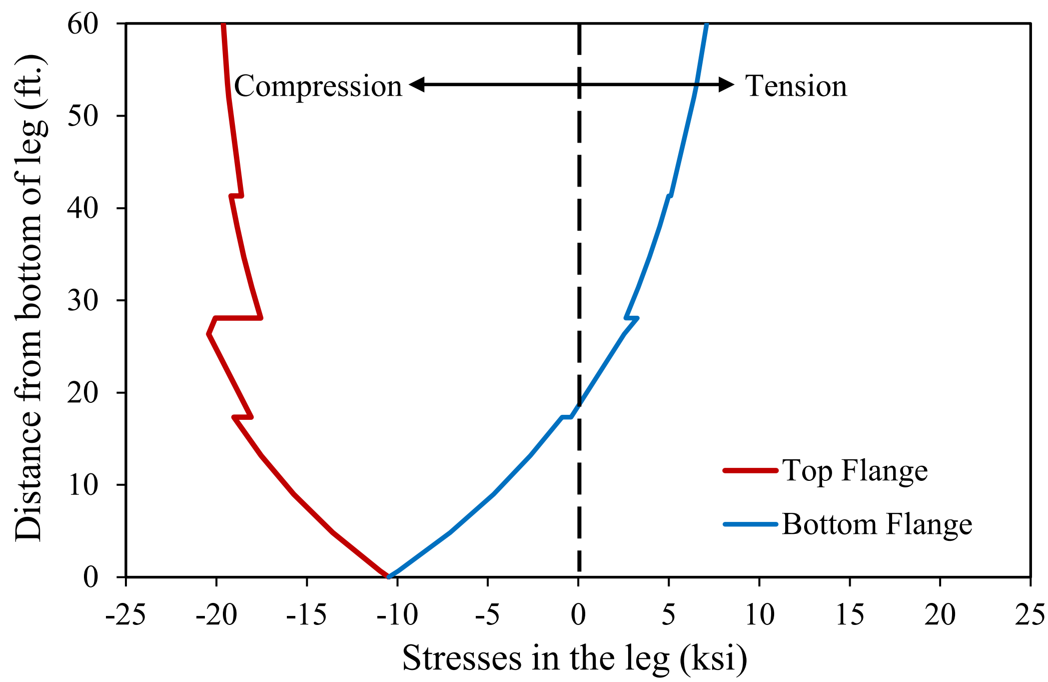


Figure 11. Dead Load and Live Load stress in the frame legs.

    - A free body diagram of the change in direction of the leg flanges at the top of the shoe (Figure 12) shows that a tension force must be resisted at this location to counteract the compression forces in the flange through its change of direction. At 11¾” wide by ¾” thick, the shoe tension tie (labeled a transverse stiffener on the design drawings) in this location was larger than the other stiffeners in this region of the leg, indicating a design function beyond just stiffening the web plate. As this element is required to maintain equilibrium in the shoe region, the result is a tension element in a component typically considered to be in full compression.

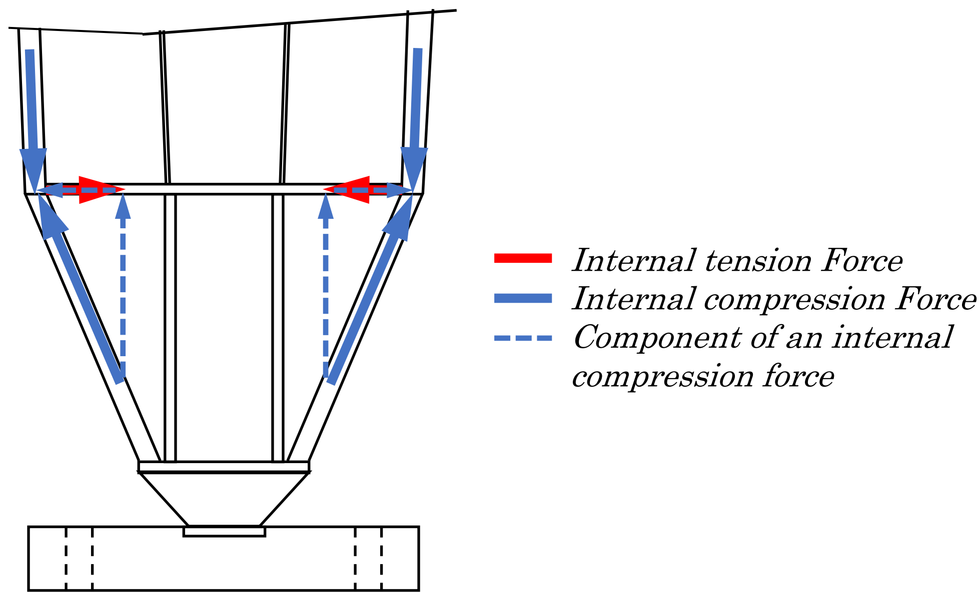


Figure 12. Detailing of leg shoe region with free body diagram of flange node at top of shoe.

- Since portions of the non-load path redundant steel frame legs were in tension, they were fracture critical members, and subject to hands-on inspection.

- However, it was unclear whether the frame legs were consistently considered FCMs and subjected to hands-on FCM inspections.
    - Starting in 2011, the inspection reports included a “Fracture-Critical Identification Framing Plan” (Figure 13), prepared by Wilbur Smith Associates, Inc., that identified the floorbeams and portions of the frame girders (those portions likely to be in tension due to flexure) as fracture critical members, but did not similarly identify the frame legs. This diagram was updated in 2015 without changes to the identification of the frame legs. The 2015 diagram was included in inspection reports through the 2021 bridge inspection.
    - Through 2015, the inspection reports included a narrative section titled “Fracture Critical Members and Intersecting Welds” that included notes on the inspection of the frame legs.
    - In 2016, CDM Smith prepared a “Fatigue and Fracture Bridge Inspection Plan” for the Fern Hollow Bridge on behalf of the City of Pittsburgh. This plan identifies the fracture critical members to be inspected, access requirements for the inspection, and notes describing the fatigue-sensitive details to monitor during

the inspection. In this procedure, only the rigid frame girders and floorbeams are identified as the fracture critical members to be inspected (but not the frame legs), consistent with the identification framing plan developed in 2011. The 2016 inspection plan document is not referenced in any of the subsequent inspection reports, making it unclear whether inspectors referenced the plan when preparing for those inspections.

- Starting in 2017, fracture critical member inspection notes were not included in the report narrative in favor of printed output from the PennDOT BMS2 system showing the fracture critical member data items reported in the system. In these reports, four member details are described: frame girder intersecting welds, frame girder transverse stiffeners, floorbeam "Hoan-like detail", and floorbeam "cut-short flanges". The description of the frame girder members in the inspection notes and in the BMS2 Coding Manual (PennDOT Publication 100A) do not provide sufficient detail to determine if they include both the frame girders and the frame legs. These notes also lacked sufficient detail to describe how the members were to be inspected nor verify whether the members were inspected in accordance with the FCM procedures.
- FCM inspections that did include the frame legs focused on two locations where welds connecting the longitudinal stiffeners to the leg intersected with splices in the web plate. In these inspections, no cracks were observed, but the inspection reported section loss in the frame leg web. It was not apparent that the shoe tension tie plate was subjected to hand-on inspection, despite it being a steel member in tension, the failure of which could lead to partial or total collapse of the bridge.

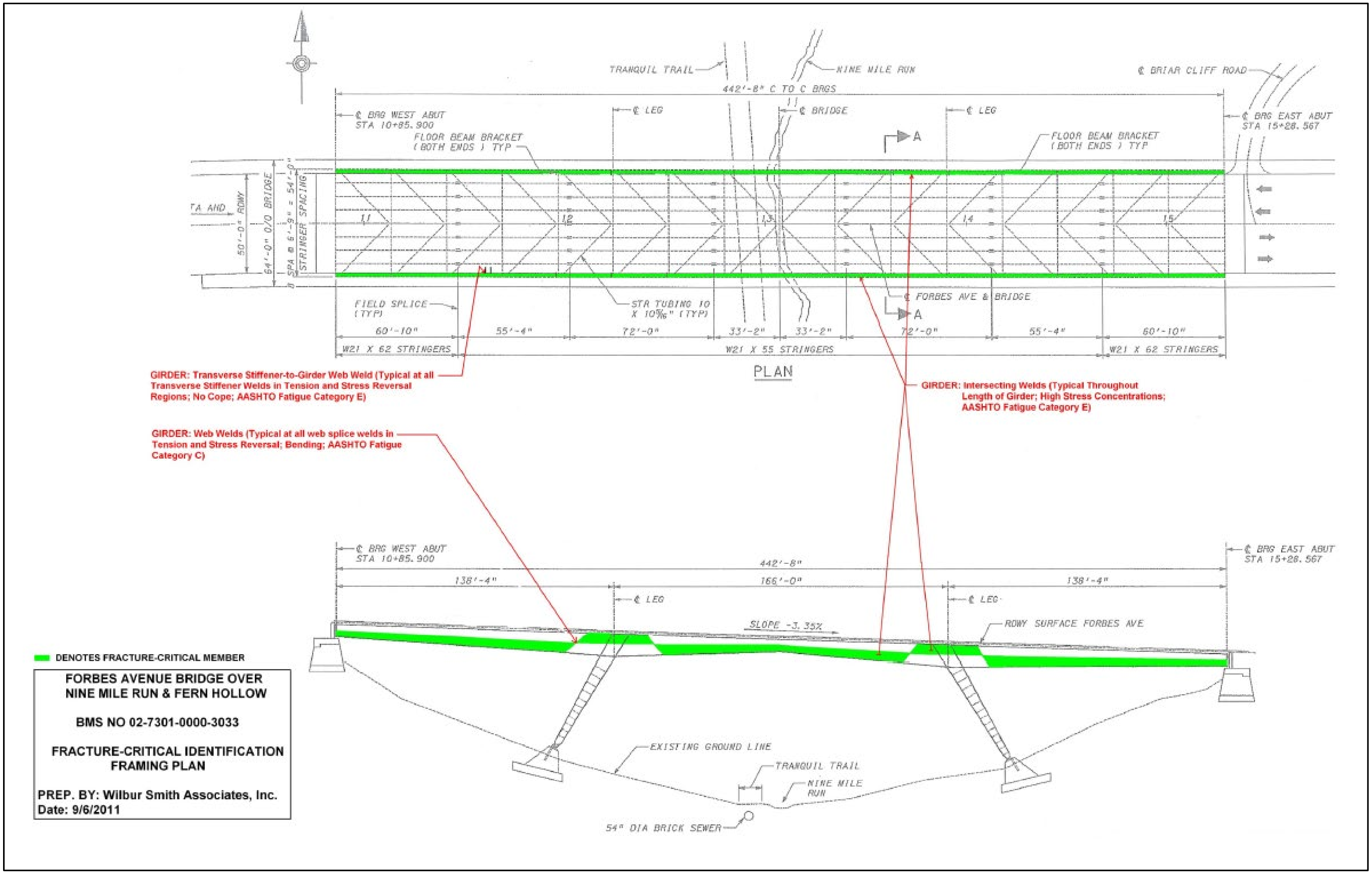


Figure 13. Fracture Critical Identification Framing Plan developed in 2011 and included in the 2011, 2013, and 2015 inspection reports

Fracture Critical Member Inspection Procedures

FHWA reviewed available fracture critical member inspection procedures and inspection reports to evaluate whether fracture critical member procedures were effective, properly documented, and followed.

- The "Fatigue and Fracture Bridge Inspection Plan" prepared for the bridge in 2016 describes the members considered fracture critical (excluding the rigid frame legs, as described above), provides instruction on means to access the FCMs, and identifies fatigue-prone details on the bridge. The plan did not address corrosion and section loss on FCMs. While the plan included a section titled "Inspection Procedures", that section only described access requirements, and the plan does not otherwise describe what inspection methods are to be employed to properly assess the condition of the FCMs.
- While the plan does include a section titled "Bridge Testing Recommendations", the procedures do not describe when non-destructive evaluation (NDE) methods are to be employed, what those methods are, nor does it prescribe a frequency for those examinations. FHWA's review of previous inspection reports found the only instance of NDE being performed was in the September 2005 inspection and the use of dye-penetrant testing on locations of cracking in the floor beam connection plate welds. These cracks were not mitigated, and future inspections did not document use of dye penetrant testing to monitor propagation.
- Thus, though cracks lengths were measured and reported from the 2016 inspection onward, it is unclear how the cracks in the rigid frame girder web at the tops of connection plates for bolting the floorbeams to the girder were being measured. Surface breaking steel crack propagation is difficult to accurately measure without use of NDE methods such as magnetic particle or dye penetrant testing. Photos and notes have no indication of use of NDE.
- There is a discrepancy in the reporting of floorbeam connection crack lengths between the September 2017 and later inspections. The September 2017 inspection reported in the crack width table (PDF pg. 83) that the cracks had propagated (for instance, growing from ⅝" to 1" long between the 2016 and 2017 inspections). This was revised on future inspections to show that there was no change in 2017 and the reports did not clarify the reason for the revision.
- As the "Fatigue and Fracture Bridge Inspection Plan" focused on fatigue-prone details, subsequent inspection reports prepared between 2017 and 2021 provided the above-described detailed measurements of the length of identified fatigue cracks, primarily at the girder-to-floorbeam connection plate weld. Corrosion-related section loss, where identified, was only characterized in terms of estimated percentage of section lost or in the measured size of perforations due to 100 percent section loss. Remaining section thicknesses were not measured.
- It is unclear from the inspection notes whether all areas of section loss on the frame legs were accessed and measured, particularly at the mid-height of the leg. Photos indicate hands-on access was achieved from the ground at the base of the leg and from the under-bridge inspection crane at the top of the leg near the connection to the rigid frame girder.

- Photos in a majority of the inspections do not show evidence of any cleaning of steel to facilitate accurate section loss measurements nor was evidence of cleaning found on the wreckage of the bridge. MBE Articles 4.8.1.2 and 4.8.3.1 state rust scale needs to be "*removed down to base metal*" to achieve this, typically with the use of grinders, hammers, or wire brushes. Numerous photos show laminar corrosion still present in the legs. Photos and notes from 2013 indicated some cleaning was performed around through holes, and inspectors applied orange spray paint to the areas, likely to highlight the edges of the through holes.
- Section loss measurements for the frame legs focused only on the size of web through holes, and descriptions of 100 percent section loss from portions of the transverse stiffeners. Despite visual evidence that the shoe tension tie plate was experiencing section loss, none of the reports reviewed included a measurement of remaining section of that element. MBE Article 4.8.3.1 states "*sufficient measurements should be taken to allow the evaluation of the effect of the losses on member capacity*." Remaining thickness measurements would be required for evaluation and load rating analysis to effectively assess the safe load carrying capacity of the bridge in its reported condition.

While fracture critical member inspection procedures were in place for the Fern Hollow Bridge and were followed with regards to visual inspection rigor of those members identified as FCMs, the procedures were inadequate in that they focused solely on the identification and monitoring of fatigue-induced cracking and did not address other deterioration modes that could lead to member failure.

Other Special Inspection Scope and Procedures

FHWA reviewed available interim inspection reports (from 2014, 2016, 2018, and 2020) from the Special (Other) inspections, required per PennDOT inspection policy due to the condition of the bridge, to evaluate their scope and the clarity of the procedures to be used therein.

- The report from the first interim inspection in 2014 noted that the scope of those inspections was to be focused on the frame legs. Subsequent inspection reports didn't explicitly state the scope, but included inspection notes carried over from previous inspections with notes for elements other than the frame marked as "Not evaluated during this inspection."
- Similar to the routine and FCM inspections, section loss due to corrosion was described in terms of measured dimensions of web perforations and portions of 100 percent section loss from transverse stiffeners. There was no evidence of cleaning to facilitate accurate measurement. There were no measurements of remaining section of the shoe tension tie.
- The cables installed in 2008 to supplement the cross braces were within the scope of the Other Special inspections, primarily with respect to their tightness. Notes from the September 2013 inspection indicated the cables were "loose and moved freely." It was documented in maintenance records that the cables were then tightened in July 2014, and future inspections indicate the cables are tight and in good condition. It is unclear how adequate cable tension was assessed by the inspectors or addressed by those performing maintenance.

Inspection Access

- The means of access for hands-on inspection was indicated in all reports to have been an under-bridge inspection vehicle with approximately 60 ft. horizontal reach. This inspection vehicle would have provided adequate access to provide hands-on inspection for the frames and floor beams based on the 64 ft. out-to-out width of the bridge. In the September 2017 reports and later, an Aspen UB-60 was documented. UB-60 was manufactured by Aspen Aerials, previously known as Reach All.
- Based on online literature reviewed for this report, the Gross Vehicle Weight (GVW) of the Pittsburgh Rigging Aspen UB-60 documented as being used for the inspections is 32 Tons or greater.
- The posted weight for the bridge was established at 26 Tons.
- It is not clear whether an analysis was performed to demonstrate that the bridge had adequate live load capacity for the UB-60 or if the inspection firms understood their inspection vehicle was over the posted load.

Inspection Notes and Documentation

The MBE, in Article 4.7, emphasizes the importance of providing clear and detailed notes and sketches in inspection reports, both for future comparisons of condition and to support the determination of the safe load carrying capacity of a bridge. Inspection forms are to be organized in a systematic manner and contain sufficient photographs and sketches to be fully interpreted by office personnel for the purpose of evaluating remaining capacity and inspector work recommendations.

FHWA reviewed available inspection reports to evaluate their effectiveness in conveying the progression of the conditions that evidence suggests led to the bridge's collapse.

- The format of the inspection notes changed over the period of the reports reviewed:
    - Between 2005 and 2016, each report contained a narrative portion with detailed notes under separate headings for each bridge component (deck, superstructure, etc.). These reports would also include bridge management system output showing how these notes were logged in the BMS database.
    - Between 2017 and 2020, there was no narrative portion to the inspection reports, but detailed output from the BMS, including inspection notes (separated by component) were included in the inspection reports.
    - In the final inspection in 2021, the report format returned to a detailed narrative supplemented by BMS output.
- Sketches were provided in the inspection reports for the near and far faces of both bents, as well as a framing plan sketch including location and extent of deterioration to the main girder, floor beams, and stringers. Sketches were limited in detail and hand drawn through the 2014 inspection. In 2015, sketches of the cross braces and legs were developed in computer-aided design (CAD) software and later inspection updated those drawings with color-coding to delineate changes in deterioration between inspections.

- The inspection narrative used consistent language to describe the location and extent of deterioration. Where appropriate, the narrative referenced inspection photos that illustrated the condition being described in the narrative. This was sufficient to chronicle the continuing deterioration of the frame legs, though (as described above) insufficient to effectively support evaluation and load rating efforts.
- Appendix A of this report documents FHWA's review of the inspection notes from the 2005 to 2021 inspection reports, focusing on descriptions of the frame legs and cross bracing.

Superstructure Condition Inventory Data

FHWA reviewed NBI data submitted annually by PennDOT on behalf of the City of Pittsburgh to evaluate the appropriateness of reported condition data as it compares to conditions recorded in the inspection report narratives. Bridge inventory information collected for every bridge open to public traffic is reported to FHWA in accordance with NBIS reporting requirements for inclusion in the NBI. This data is used by FHWA and bridge owners to classify bridges as to their function and condition, identify funding needs, and monitor compliance with the NBIS.

Overview – General Condition Ratings

General bridge condition ratings, encapsulating the results of bridge inspections, are recorded in three data items in the NBI, described in the FHWA Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges (otherwise known as the Coding Guide) – Item 58 for the condition of the deck, Item 59 for the superstructure, and Item 60 for the substructure. These summary "component" condition ratings are assessed on a 0 to 9 scale, where 0 represents a failed condition and 9 is excellent condition. The assessment of the condition ratings is guided by the language in the Coding Guide shown in Figure 14. Since the Fern Hollow Bridge is a steel rigid frame, the girder and leg portions were all considered part of the superstructure.

Code	Description
N	NOT APPLICABLE
9	EXCELLENT CONDITION
8	VERY GOOD CONDITION - no problems noted.
7	GOOD CONDITION - some minor problems.
6	SATISFACTORY CONDITION - structural elements show some minor deterioration.
5	FAIR CONDITION - all primary structural elements are sound but may have minor section loss, cracking, spalling or scour.
4	POOR CONDITION - advanced section loss, deterioration, spalling or scour.
3	SERIOUS CONDITION - loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	CRITICAL CONDITION - advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1	"IMMINENT" FAILURE CONDITION - major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	FAILED CONDITION - out of service - beyond corrective action.

Figure 14. Item 58, 59, and 60 coding guidance (FHWA "Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges (December 1995)".

The Coding Guide also provides the following guidance in applying the condition codes to the overall component when there are localized defects:

> *"Condition codes are properly used when they provide an overall characterization of the general condition of the entire component being rated. Conversely, they are improperly used if they attempt to describe localized or nominally occurring instances of deterioration or disrepair. Correct assignment of a condition code must, therefore, consider both the severity of the deterioration or disrepair and the extent to which it is widespread throughout the component being rated."*

The Coding Guide emphasizes the overall characterization nature of the condition coding, and that localized defects should not typically drive the condition rating of an entire component. However, the severity and extent of the defect must be considered. For example, on a non-load path redundant member, the overall effect of a localized defect may be severe, in that the localized defect on one member could lead to the failure of the entire component.

This is illustrated in the BIRM, in Topic 4.2 "Condition and Appraisal," which further clarifies how to account for severe, but localized, deterioration,

> "*Although the FHWA Coding Guide states that it is improper to use the condition codes to describe localized instances of deterioration or disrepair, it also states that the inspector must consider both the severity and extent of the deterioration. With this in mind, there are occasions when a severe, localized condition affects the structural capacity of a component member. It is*

important to recognize that the coding applies to all primary members of a component. Therefore, localized conditions that impact the structural capacity of just one member can impact the overall performance of the entire component. The effect on structural capacity is dependent upon several factors including the type and extent of the deterioration, as well as the location along the member. An inspector may need to discuss the observed condition with an engineer to make this determination. When these situations occur, it is appropriate to assign a lower component condition rating for that component from a safety perspective and is in keeping with the intent of the National Bridge Inspection Program."

The BIRM goes on to tie severe localized deterioration to the concept of critical findings, as defined in 23 CFR 650.303. It presents two ways in which a State's policies and procedures might handle the coding of the condition rating when critical findings exist.

1. The first approach is to rate the entire component lower to reflect the critical finding and then to adjust the component condition rating once the critical finding has been resolved.

2. The second approach is to "rate to the average" and utilize the rating to characterize the overall condition of the component without rating down to account for the severe localized deterioration or critical finding. This approach is dependent on critical findings being identified and acted on regardless of component condition rating.

Either approach is acceptable, but it is imperative under either method that critical findings are clearly communicated by the inspectors and promptly addressed by the Owner. The BIRM states that, "The coding of NBI condition items should be viewed as important, but secondary, to the recognition of and follow-up on critical findings."

Fern Hollow Bridge General Condition Data

On the inspection reports, Item 59 superstructure condition was assessed as follows:

- September 2005 through September 2009, Item 59 = 5 (Fair Condition).
- September 2011 through September 2021, Item 59 = 4 (Poor Condition).

The inspectors utilize the terms "poor to critical" during inspections from 2018-2020 to characterize the condition of the frame legs, which indicates they realized the severity of the corrosion, section loss and holes in the webs, flanges, and stiffeners of the legs; however, this is not reflected in the Item 59 coding.

Based on review of the inspection information, the Coding Guide, and the BIRM, a more appropriate and defensible code for the overall superstructure condition would have been 3 – Serious Condition. Serious condition aligns with the effect severe loss of section and deterioration may have on the lower frame legs, which are primary structural elements of the superstructure itself. While the girder portions of the frames were in fair condition with minor section loss, application of the above BIRM guidance in assessing severe localized deterioration in the lower frame legs would lead to rating the entire superstructure component lower. The frame legs are non-redundant, and a reduction in capacity from the section loss in the lower frame leg would affect the overall performance of the superstructure. Additionally, description of condition rating of 3 indicates that local failures are possible. Due to the significant localized section loss and web perforations, local failures should have been seen as a possibility.

The advanced deterioration of the legs could have also warranted a condition rating of 2 – Critical Condition. Critical condition reflects that unless closely monitored, it may be necessary to close the bridge until corrective action is taken. As the legs were documented to continue to deteriorate at an accelerated rate, corrective action was needed to maintain proper support and structural stability.

The section loss to the frame legs was not identified by the inspectors as a critical finding. Regardless of the Item 59 condition rating, the severity of the section loss to the web and web stiffeners at the base of the legs warranted a critical finding due to the location on a non-redundant member.

Overview – Element Level Bridge Condition Data

In addition to the component condition ratings, another measure of condition used during bridge inspections is an assessment of element level condition data. Element-level inspection assesses the condition of the elements that make up the bridge (e.g. concrete deck, steel girders, steel floorbeams, and concrete abutment) and is reported as quantities of each element found to be in each of 4 Condition States. This is intended to provide data that is more granular than component condition data and better support the deterioration models central to modern asset management systems. The 2012 MAP-21 legislation made the collection of element level bridge inspection data for submittal to the NBI a requirement for all bridges on the National Highway System (NHS). Forbes Avenue is on the NHS, thus element-level condition assessment was required on the Fern Hollow Bridge.

Element level bridge assessment is made based on four condition states (CS 1 to CS 4), with CS 1 indicating no defects (Good condition) and CS 4 indicating the presence of severe defects warranting structural review (Severe condition). Each successive condition state is indicative of progressing deterioration, as described in the AASHTO *Manual for Bridge Element Inspection*. Figure 15 shows the descriptions for steel elements. Note that any defect in CS 4 to be in severe condition and "*warrants a structural review to determine the effect of serviceability of the element or bridge*."

Defects for Steel Elements

Defects	CS 1 GOOD	CS 2 FAIR	CS 3 POOR	CS 4 SEVERE
Corrosion (1000)	None.	Freckled rust. Corrosion of the steel has initiated.	Section loss is evident or pack rust is present but does not warrant structural review.	The condition warrants a structural review to determine the effect on strength or serviceability of the element or bridge; OR a structural review has been completed and the defects impact strength or serviceability of the element or bridge.
Cracking (1010)	None.	Crack that has self-arrested or has been arrested with effective arrest holes, doubling plates, or similar.	Identified crack that is not arrested but does not warrant structural review.	
Connection (1020)	Connection is in place and functioning as intended.	Loose fasteners or pack rust without distortion is present but the connection is in place and functioning as intended.	Missing bolts, rivets, or fasteners; broken welds; or pack rust with distortion but does not warrant a structural review.	
Distortion (1900)	None.	Distortion not requiring mitigation or mitigated distortion.	Distortion that requires mitigation that has not been addressed but does not warrant structural review.	
Settlement (4000)	None.	Exists within tolerable limits or arrested with no observed structural distress.	Exceeds tolerable limits but does not warrant structural review.	
Scour (6000)	None.	Exists within tolerable limits or has been arrested with effective countermeasures.	Exceeds tolerable limits but is less than the critical limits determined by scour evaluation and does not warrant structural review.	
Damage (7000)	Not applicable.	The element has impact damage. The specific damage caused by the impact has been captured in CS 2 under the appropriate material defect entry.	The element has impact damage. The specific damage caused by the impact has been captured in CS 3 under the appropriate material defect entry.	The element has impact damage. The specific damage caused by the impact has been captured in CS 4 under the appropriate material defect entry.

Figure 15. Steel element defect and Condition State descriptions (AASHTO MBEI, 2nd Edition).

Though not required for element level data submitted to the NBI, owners can collect another layer of element data that assigns quantities in each condition state to a particular defect – for instance, Defect 1000 – Corrosion. The collection of this additional data can help further inform asset management budgeting models and decisionmakers on potential work actions that might be required to mitigate defects identified in inspection.

Fern Hollow Bridge Element Level Condition Data

Element-level inspection data, including defect-level data, was collected during the inspections performed in September 2019, 2020, and 2021. The frame legs were inventoried as Element 202 - Steel Column, for which conditions are assigned on an each (EA) basis. In all three of these inspections, all four legs were assessed to be in CS 4 for the defect of corrosion, which is an appropriate condition assessment. As described above, CS 4 indicates that the condition warrants structural review, or that the defect has been reviewed and does impact strength or serviceability of the bridge. No documentation was found in the provided reports indicated that a structural review was performed related to these findings.

Maintenance Recommendations:

While it is a requirement of the NBIS for State DOTs to address critical inspection findings, the assignment and performance of maintenance work recommendations stemming from bridge inspections is not a Federal requirement. As such, in Pennsylvania, maintenance work items recommended by bridge inspectors on local agency-owned bridges are carried out by the bridge owner. PennDOT utilizes a maintenance prioritization system that assigns a priority of 0 to 5 to each work recommendation originating from a bridge inspection. Guidance for coding maintenance prioritization, including detailed examples of Priority 0 to 5 items, is provided in PennDOT Publication 100A under Item IM05 (pg. 3-331). Figure 16 is an excerpt from PennDOT Publication 100A describing the coding of this item.

Priority 0 items are critical issues that require immediate resolution to preserve safety, within a timeframe not to exceed 7 days. Priority 1 items are high priority issues that require a plan of action to be established for addressing the issue within 6 months. The process for addressing critical and high priority maintenance items is in PennDOT Publication 238, Section 2.13.2, including PennDOT oversight for critical or high priority maintenance items. In this process, the PennDOT District must review and approve both the maintenance prioritization of the finding and the plan of action for addressing Priority 0 or 1 maintenance recommendations on local agency-owned bridges. PennDOT then tracks the findings to ensure completion of the maintenance actions within designated timeframes. Per note 2) for Item IM05, the action timeframe for a Priority 2 maintenance item is the interval to the next routine inspection (24 months for the Fern Hollow Bridge).

Coding:

		Short Definition	Action Timeframe
0	**CRITICAL**	Immediate response required	(within 7 days)
1	**HIGH PRIORITY**	As soon as work can be scheduled	(within 6 months)
2	**PRIORITY**	Review work plan and re-prioritize schedule.	(routine inspection interval)
3	**SCHEDULE**	Add to scheduled work.	(Add to schedule)
4	**PROGRAM**	Add to programmed work	(when funds are available)
5	**ROUTINE**	As per existing maintenance schedule.	(within the next work cycle)

Notes:

1) The District Bridge Engineer (and owner for non-PennDOT bridges) must be advised of conditions that warrant a Priority code 0 or 1 Flexaction work candidate, and must accept this coding before Item 1A07, Inspection Status, is changed to Approved. See Publication 238 Sections 2.13 and 2.14 for specific guidance and required actions for Priority Codes 0 and 1.
2) The action timeframe for a Priority 2 repair is the same as the routine inspection interval for the bridge (e.g., a bridge with a 48-month routine inspection interval will have a 4-year action timeframe but a bridge with a 24-month routine inspection interval will have a 2-year action timeframe).
3) All Flexactions must be recorded and input into BMS2 regardless of assigned Priority code.
4) If priority changes occur for a maintenance action that has not been sent to SAP, the previous priority(ies) along with original supporting information must be documented in the notes field.

Figure 16. Item IM05 (Maintenance Recommendation Priority) coding guidance (PennDOT Publication 100A).

Table 1 shows a history of Priority 0 through 2 maintenance items from the provided inspection reports pertaining to structural issues or load postings, including any dates of documented completion. Over the timeframe of 2005 to 2021, four Priority 0 items were completed: two were related to load posting signage, one was related to removal of a light pole with severe corrosion at the base, and the fourth was related to removal of the cross brace that was at risk of falling off the bridge (this was originally a Priority 1 item). At no time were repairs to the frame legs, to address the deterioration affecting the capacity of the bridge, made a Priority 0 or 1 recommendation.

The only completed Priority 2 item was re-tensioning the retrofit cables. No other Priority 2 recommendations were documented as being completed over this timeframe, although many were identified repeatedly, such as repairs to address the web and web stiffener section loss on the frame legs, zone painting of the frame leg areas exposed to runoff, and flushing the drainage system.

Table 1. Fern Hollow Bridge Priority 0-2 structural or load posting related maintenance items, from 2005-2021 inspection reports

Recommended Maintenance Item Description	Priority	First Year Identified	Additional Years Identified	Documented Completion
Repair/replace stiffeners and web on frame legs	2	2007	2009-2021	
Repair cross bracing on both frame legs	2	2005	2007-2021	
Re-tension cables on legs	2	2009	2011-2014	7/25/2014
Extend PVC "weepholes" in deck to drain below superstructure	2	2005	2007-2021	
Paint superstructure areas exposed to leakage, primarily the frame legs[a]	2	2007	2014-2021	
Drill crack arrest holes in FB/girder connection plate cracks	2	2015	2016-2021	
Clean and flush deck scuppers (drains)[b]	2	2017	2018-2021	
Repair/replace lower cross frame at Bent 1 which is nearly severed at connections.	1	2017	3/2018	1/4/2019 cross frame was removed
Remove or replace defective light pole on deck.	0	2009		By 2011 inspection, all light poles were replaced.
Repair/replace lower cross frame at Bent 1 which has become severed. (priority raised to 0)	0	9/2018		1/4/2019 cross frame was removed
Add "bridge" placards to all postings	0	2015		Before 2016 inspection
Add "distance ahead" placards to all postings	0	2020		9/11/2020

[a]This recommendation was initially identified as a Priority 2 in 2007. It then appeared as a Priority 4 in 2009-2013, before being raised to a Priority 2 again in 2014.
[b]This recommendation appeared in numerous inspection reports as a Priority 3, not until 2017 was it elevated to Priority 2.

Load Rating Recommendations

FHWA reviewed available inspection reports to determine what level of structural evaluation was recommended in response to the progression of section loss in the rigid frame legs identified during bridge inspections.

- For September 2005 through September 2011 inspections, no load rating was recommended by inspectors. Load rating data reported from these inspections were based on analyses performed in 2000 and 2003
- On the September 2013 inspection, a load rating was recommended and subsequently performed to "*assess the stability of the structure assuming that the cross braces are nonfuctional*". However, the section loss resulting from the corrosion on the frame legs was not considered in this load rating.
- Between the September 2014 and September 2021 inspections, no re-rating or change to load posting was recommended. In the report from the 2021 routine inspection, under the "Load Rating Summary" it was noted that "*Because the condition of the main load carrying members has not changed significantly, the 2014 Load Rating Analysis is still valid.*" It was not made clear whether the inspectors made this assessment, or if it was the judgement of the engineer responsible for the 2014 load rating.

Load rating is discussed in more detail in the Review of Fern Hollow Bridge Load Ratings section of this report.

Load Posting

FHWA reviewed available inspection reports to identify inspector observations, findings, and recommendations related to posted load restrictions on the bridge.

- PennDOT BMS2 shows a 26 Ton posting implementation date of July 8, 2014, which was confirmed in the 2014 inspection report.
- In the September 2015- a critical (Priority 0) maintenance item was recommended for adding the "bridge" placard to the weight postings and advanced postings. This is necessary for the enforceability of posting and compliance with Pennsylvania state laws and standards. The bridge placard was noted as being added in the 2016 inspection.
- September 2018- a high (Priority 1) maintenance item was recommended to adjust and re-align the advanced posting on the far approach due to impact damage. This was recorded as being completed on the 2019 inspection.
- September 2020- a critical (Priority 0) maintenance item was recommended for adding distance placards to the load posting signage. This is necessary for enforceability of posting and compliance with state laws and standards. The distance placards were noted to be added on the 9/2021 inspection.

SUMMARY OF FINDINGS

- The inspectors did not identify a critical finding for the severe local deterioration to the frame legs, as would be required by the National Bridge Inspection Standards and indicated by the severity, extent, and location of the deterioration observed in multiple inspections.
- The worsening condition of the legs, including growing web through-holes, complete section loss of some transverse and longitudinal stiffeners, and most importantly the section loss in the shoe tension tie, was not measured nor considered by the inspectors to necessitate a re-rating or additional analysis, despite the increased severe section loss to the legs demonstrating a change in the capacity of the frame.

- The condition rating of Item 59- Superstructure was coded by the inspectors as a 4 (poor condition) from 2011 to 2021. Inspection reports from 2018-2020 characterize the frame legs as "poor to critical" condition due to the corrosion, section loss, and through holes of the webs and stiffeners, indicating they understood the severity of the condition. A more appropriate coding of Item 59 would have been 3 (serious condition) or 2 (critical condition), reflecting the severity of the condition of the non-redundant frame legs. The coding language for a condition rating of 3 indicates that local failures are possible.
- Although perforations and holes in the steel were identified, the extent, severity, and location of remaining section on the frame legs was not sufficiently documented as required by the AASHTO Manual for Bridge Evaluation. Detailed information regarding remaining section, necessary for a load rating, was not provided in the notes and sketches. Report photos did not show adequate cleaning of the laminar corrosion on the web to provide accurate remaining section measurements.
- At no time were repairs to the frame legs to address the significant section loss to the web and web stiffeners assigned high priority (PennDOT code 0 or 1) by inspectors. Instead, these repairs were assigned as Priority 2 maintenance recommendations for the 15 years prior to the bridge collapse. Priority 2 maintenance recommendations, per PennDOT policy, have a timeframe to address of the routine inspection interval or 24 months. Only one Priority 2 maintenance recommendation was documented as completed by the City of Pittsburgh over the timeframe of the review (2005-2021), which was re-tensioning the retrofit cables on the frame legs in July 2014. No repairs or preservation activities were documented as being completed on the frame legs other than installation of the retrofit cables in 2009 and retightening of the cables in 2014.
- FCM inspection procedures did not identify the frame legs as FCMs. The steel frame legs have tension elements and are non-redundant and therefore should have been treated as FCMs.

REVIEW OF FERN HOLLOW BRIDGE LOAD RATINGS

BRIDGE LOAD RATING AND POSTING OVERVIEW

A bridge's load rating establishes the maximum vehicular (live) load that it can safely carry. In the absence of severe deterioration, the load rating is based on the as-built capacity of the bridge and is used by the bridge's owner to determine whether to approve requests to move loads larger than the established legal loads over the structure. If the requested load exceeds the load rating of a bridge, the load may be redirected to other routes with bridges having sufficient capacity (as demonstrated through a load rating) to safely carry the permitted load. In the case where deterioration or other deficiency indicates that the design capacity can no longer be assumed, the load rating (recalculated to consider current bridge conditions) serves as both a measure of remaining capacity and a tool to identify legal loads that can still use the bridge. If the rating indicates that a class of legal vehicles cannot be carried safely, the NBIS requires the bridge to be posted to limit truck weights to no more than the heaviest vehicle that it can safely carry[25].

The requirement to rate the safe load-carrying capacity of each bridge, and to post or restrict legal and routine permit loads that exceed the load rating, has been in the National Bridge Inspection Standards since its inception in 1971. At the time of the collapse, bridge load rating in the United States was guided by the AASHTO *Manual for Bridge Evaluation* (MBE), First Edition, published in 2008[26]. The MBE describes three methods for calculating load ratings, the selection of which is prescribed by the age of the bridge, the method used to design the bridge, and agency policy (Table 2).

Table 2. Load Rating Methods

Load Rating Method	Where Used
Load and Resistance Factor Rating (LRFR)	All bridges designed using Load and Resistance Factor Design (LRFD) after October 1, 2010 Optional for bridges designed by any method prior to October 1, 2010
Load Factor Rating (LFR)	Optional for bridges designed by any method prior to October 1, 2010
Allowable Stress Rating (ASR)	Optional for bridges designed by any method prior to October 1, 2010

For each method, the MBE establishes the live load models to be considered in the rating and the load factors used to reflect the uncertainty inherent in the load calculations. The three methods

[25] 23 CFR 650.313(l)(1)
[26] Incorporated by reference at 23 CFR 650.317(b)(1). Between the time of the collapse and when this report was submitted to NTSB, the National Bridge Inspection Standards were revised and finalized in May 2022. At the time of this report, the NBIS incorporates the AASHTO Manual for Bridge Evaluation, Third Edition, with 2019 and 2020 interim revisions, incorporated by reference at 23 CFR 650.317(a).

differ most notably in the application of these load factors, with the older methods being retained to allow ratings to be consistent with the methodology used to design the bridge. According to National Bridge Inventory data, Load Factor Rating is the most common rating method in use at the time of this writing.

All three methods use the same basic formulation to calculate the rating:

$$RF = \frac{C - A_1 \cdot DL}{A_2 \cdot LL(1 + I)}$$

Where:

$RF =$ The rating factor, calculated as a fraction of the weight of the rating vehicle under consideration

$C =$ The capacity of the member under consideration which, depending on the rating method, might be reduced to account for uncertainty

$DL =$ Dead load force effect (axial load, moment, or shear) on the member due to the weight of the bridge itself and other permanent loads, such as asphalt wearing surface

$LL =$ Live load force effect on the member from the rating vehicle model under consideration

$I =$ Impact factor, to account for dynamic effects from heavy trucks, expressed as a percentage

$A_1 =$ Factor to account for uncertainty in dead loads

$A_2 =$ Factor for to account for uncertainty in live loads

Force effects (tension, compression, moment, or shear) on a member will be established using structural analysis, performed either by hand calculation or using computer techniques. For the calculation of member capacity, the MBE refers to the AASHTO bridge design specifications relevant to the rating method selected (Table 3). As member capacity is dependent on the geometric and material properties of the structural section (i.e., yield strength, area, moment of inertia), the member capacity calculation must take into account loss of section due to deterioration.

All bridges are rated for either the HS-20 (ASR and LFR) or HL93 (LRFR) live load models, as those loads match the standard loads used in bridge design. Additional AASHTO or state-specific load models will also be evaluated as deemed necessary to ensure that the rating considers the effects of all vehicles that can travel without restriction on the roadway crossing the bridge.

Table 3. Load Rating Methods, Capacity Calculation Specifications

Method	Design Specification for Capacity Calculation
Load and Resistance Factor Rating (LRFR)	AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications (current edition)
Load Factor Rating (LFR)	AASHTO Standard Specifications for Highway Bridges (17th Edition)
Allowable Stress Rating (ASR)	AASHTO Standard Specifications for Highway Bridges (17th Edition)

Ratings are calculated at two levels, inventory and operating, which are defined in the MBE as follows:

- *Inventory Rating* - Load ratings based on the Inventory rating level allow comparisons with the capacity for new structures and, therefore, results in a live load which can safely utilize an existing structure for an indefinite period of time.
- *Operating Rating* - Load ratings based on the Operating rating level generally describe the maximum permissible live load to which the structure may be subjected. Allowing unlimited numbers of vehicles to use the bridge at the Operating level may shorten the life of the bridge.

A calculation of the inventory and operating rating will use the same values for force effects, dead load factor, A_1, and capacity but will vary in the application of the factor for live load, A_2. In the simplest terms, a rating factor value equal to or greater than 1.0 indicates that there is sufficient capacity for the bridge to carry the rating vehicle, while a rating factor less than 1.0 indicates that the bridge does not have sufficient capacity to carry the rating vehicle within the safety level assumed by the rating levels (Inventory or Operating) described above. A negative rating factor indicates that the bridge's capacity is exceeded by the factored dead load. When a bridge's calculated operating rating is less than the weight of a State's maximum unrestricted legal or routine permit loading configurations (i.e., a rating factor less than 1.0 for that vehicle), the NBIS requires that the bridge be posted or restricted. Further, the MBE requires that bridges not capable of carrying a minimum gross vehicle weight of three tons must be closed.

A complete bridge load rating will include inventory and operating rating factors calculated at as many member locations necessary to identify the controlling (i.e. lowest) rating. Similarly, rating factors will be calculated for a suite of vehicle configurations in order to establish the need for posting for each configuration. Load rating results can be expressed either in terms of the rating factor or vehicle tonnage, calculated as the rating factor multiplied by the established legal gross vehicle weight of the configuration being evaluated. Generally, rating factors provide a normalized value better suited to making posting determinations, while vehicle tonnage is better suited to communicating posting values to the traveling public.

Pennsylvania bases its posting determinations on what is termed the "Safe Load Capacity" (SLC) which is defined in Chapter 4.3.2 in Part IP of Pennsylvania DOT's Publication 238 "*Bridge Safety Inspection Manual*". The SLC is determined by multiplying the operating rating by the Safe Load Capacity Reduction Factor, which additionally restricts loads based on the condition of the bridge:

$$SLC = OR \cdot f$$

Where:

$OR =$ Operating Rating

$f =$ Safe Load Capacity Reduction Factor, from Table IP 4.3.2-1 of Publication 238 (Figure 22)

Table IP 4.3.2-1: Safe Load Capacity Reduction Factors

ADTT > 500	Superstructure or Substructure		
Condition Rating	> 5	4	< 3
f	1.0	0.80	0.80

ADTT < 500	Superstructure or Substructure		
Condition Rating	≥ 5	4	≤ 3
f	1.0	0.90	0.80

Figure 17. Reproduction of Table IP 4.3.2-1 from PennDOT Publication 238.

LOAD RATING RECORDS FOR THE FERN HOLLOW BRIDGE

FHWA's review of the available records for the Fern Hollow Bridge found the following load rating analyses:

- June 2000 – Rating analysis, performed by HDR Engineering, evaluating the floorbeams and stringers under the AASHTO H-20 and HS-20 live load models and Pennsylvania's ML80 live load model using the Bridge Analysis and Rating (BAR7) program. This analysis did not calculate ratings for the rigid frame girders or legs. This analysis was documented in each of the routine bridge inspection reports between 2005 and 2013.
- September 2003 – Rating analysis, performed by Wilbur Smith Associates, supplemented the 2000 load rating by evaluating the floorbeams and stringers under Pennsylvania's TK527 live load model using the BAR7 program. This analysis also did not calculate load ratings for the rigid frame girders or legs. This analysis was documented in each of the routine bridge inspection reports between 2005 and 2013.
- October 2013 to January 2014 – Rating analysis, performed by CDM Smith, evaluating the floor beams, stringers, and rigid frame girders and legs under the H-20, HS-20, ML-80, and TK527 live load models. This rating was initiated by an "Immediate Attention" recommendation in the 2013 routine bridge inspection report to "*perform an analysis of the stability of the structure assuming that the cross braces are nonfunctional.*"

  Analysis of the girders, floor beams, and stringers was performed in the BAR7 program. The program CSiBridge was used to analyze the total structure and generate force effects under the H-20, HS-20, ML-80, and TK527 live load models, with results post-processed using spreadsheets. This analysis was first referenced in the 2014 interim bridge inspection report and documented in the 2015 routine bridge inspection report. This analysis was reported in the September 2021 inspection report as the most current load rating.

FIELD OBSERVATIONS RELEVANT TO THE LOAD RATING

In the recovery of evidence at the collapse site, thickness measurements of the asphalt wearing surface on the bridge deck indicated that it was thicker than assumed in the bridge design and load rating analyses. As the wreckage assumed the shape of the ravine, the deck slab would slide relative to the superstructure as wreckage was removed by the demolition contractor exposing the thickness of the wearing surface at various locations, so it was deemed sufficient to take photos of the wearing surface thickness instead of attempting to collect deck cores to verify thickness. Measurements were collected in four locations, two on the west side, and two of the east side of the bridge as shown in Figures 18 through 23.

The bridge was originally designed assuming a 7½" thick concrete deck slab (with 1" thick haunches at the stringers) and a 3" thick bituminous wearing surface. Bridge inventory data recorded in the 2021 routine bridge inspection report[27] (PennDOT data items 6A33 and 6A34) indicated that the wearing surface was three inches thick, last recorded on September 6, 2013 (just prior to the most recent load rating analysis). It was unclear from the inspection report if this value was measured in the field or, if so, where it was taken. Field measurements of asphalt thickness can be made by measuring exposed curb or barrier heights (and comparing to design heights) or by coring. On the west side of the bridge, the bituminous wearing surface thickness measured after the collapse varied from 5½" to 6⅝" thick. On the east side of the bridge deck, the thickness ranged from 4¾" to 5½".

Figure 18. View of demolition removal looking far to near on February 8, 2022.

Figure 19. View of demolition removal looking near to far on February 12, 2022.

[27] 2021 Routine Bridge Safety Inspection Report, Page 61

Figure 20. Reference scale of wearing surface thickness to left of center in Figure 18.

Figure 21. Reference scale of wearing surface thickness to left of Figure 20

Figure 22. Reference scale of wearing surface thickness from left side of Figure 19.

Figure 23. Reference scale of wearing surface thickness from right side of Figure 19.

Review of available records to explain the discrepancy between the asphalt wearing surface thickness assumed in the design and reported from bridge inspections (three inches thick), versus the actual asphalt wearing surface thickness measured after the bridge collapse was inconclusive. The City of Pittsburgh provided paving records demonstrating that resurfacing of Forbes Ave. occurred in July 1983, June 2000, July 2005, April and May 2009, and June and July 2017. In each case, the project specifications required that the remaining in-place wearing surface was to be milled away and replaced by three inches of new asphalt. However, because the records were based on pay items considering a range of thicknesses, FHWA could not determine how much was removed and paved back during each resurfacing operation. The photographs in Figures 18 through 23 indicate there were two distinct lifts (layers indicating placement at different times) of asphalt, but could not discern the age of the lifts. Similarly, it cannot be discerned if the original wearing surface at the time of construction was thicker than that shown on the construction plans, or if the thickness grew over time with successive resurfacing projects. Nevertheless, the discrepancy between the measured and reported thicknesses is significant in that additional dead load from the thicker wearing surface reduced the capacity available to carry vehicular live loads.

EVALUATION OF LOAD RATING ANALYSES

FHWA reviewed the previous load rating analyses for the Fern Hollow Bridge and performed independent calculations to verify the results of those analyses.

Review of Load Rating Analyses from 2000 and 2003

FHWA noted the following in its review of the 2000 and 2003 load ratings:

- The program BAR7, version 7.10, was used to rate the stringers and floor beams.
- The analyses performed of the floor beams and stringers provide a valid representation of the structural condition and capacity of those members.
- Neither of these analyses documented the determination of the capacity of the rigid frame girders and legs. As such, neither accounted for deterioration of those elements that was being documented in the inspection reports.
- The load rating summary included in the 2005 inspection report provides inventory and operating rating values for the rigid frame for the H-20, HS-20 and ML-80 live load models, but does not include calculations showing the basis for that rating.

These analyses remained the basis for the bridge's reported load rating until 2013, even as inspection reports in subsequent years described continued deterioration of these primary load carrying members. A complete load rating of a bridge is required to include an evaluation of all primary load carrying members that make up the load path and consider all deterioration that could reduce the load carrying capacity of a member. The MBE describes the complete documentation of a load rating to include all supporting computations and a clear statement of all assumptions used in calculating the load rating.

Review of the Load Rating Analysis from 2014

FHWA noted the following in its review of the 2014 load rating:

General

- The analysis used the LFR method to calculate the live load capacity of the stringers, floor beams, rigid frame girders and rigid frame legs for the H-20, HS-20, ML-80, and TK527 live load models.
- The program BAR7, version 7.13, was used to rate the stringers and floor beams. The program CSiBridge was used to perform the structural analysis of the entire structure, the results from which were post-processed in spreadsheets to compute load rating values for the rigid frame girders and legs.
- The analysis considered the dead load weight of the 7½" thick bridge deck and 3" thick bituminous wearing surface. The 3" wearing surface thickness is consistent with that documented in the September 6, 2013 inspection report as being based on a measurement made for that inspection.

- Section losses, based on measurements taken during inspection (presumed to be from the 2013 routine inspection), were considered in the analysis of the stringers, floor beams and lower portions of the frame legs though the use of equivalent material thicknesses.
- The frame leg cross bracing was not included in the analysis model as it was no longer assumed to be effective. It was not made clear in the calculations how the cable bracing installed in 2008 was accounted for in the rating.
- The controlling member and section for the load ratings was determined to be the frame leg at a section 18 feet up from bottom of the leg, measured along its length. The controlling load effect was determined to be shear.
- Based on the condition rating of the superstructure (4 – Poor) and the average daily traffic (ADT > 500), per PennDOT Publication 238 a Safe Load Capacity Reduction Factor, f, equal to 0.8 was used to calculate the Safe Load Capacity.
- The controlling operating load ratings were calculated as follows:

Table 4. Controlling RF, Operating Factor, and SLCs

Live Load Model	Operating Rating Factor	Operating Rating Tonnage	Safe Load Capacity
H-20[a]	1.79	35 Tons	28 Tons
HS-20[b]	0.92	33 Tons	26 Tons
ML-80[c]	0.95	34 Tons	27 Tons
TK527[d]	0.88[a]	35 Tons	28 Tons

[a]H-20 vehicle is 40 tons.
[b]HS-20 vehicle is 36 tons.
[c]ML-80 vehicle is 36.6 tons.
[d]TK527 vehicle is 40 tons.

- Based on these results, the bridge was posted for 26 Tons.

Load rating of the floor beams and stringers

- The analysis reiterated the dead load calculations from the 2000 load rating and considered the weight of the concrete deck and, for the floor beams, the weight of stiffeners and lateral bracing.
- Section losses were considered in the ratings of the floor beams (1/16-inch loss of thickness from the bottom flange and web) and stringers (1/16-inch loss of thickness from the top flange). These values are consistent with those reported in the September 6, 2013 inspection report.
- The capacity analysis of the floor beams and stringers was reasonable and all factors that affected the capacity of these elements at the time of the load rating calculation were appropriately considered.
- The computed operating ratings for the floor beams and stringers were all above legal weight limits. The controlling Safe Load Capacity for the stringers was calculated as 33 Tons for the ML-80 live load model, which is higher than the 26 Ton posting load.

Load rating of the rigid frame girders

- The analysis considered the dead load weight of a 3" thick wearing surface, as well as weight from miscellaneous details (sidewalks, railings, curbs, longitudinal and transverse stiffeners etc.).
- It was not clear in the documentation whether the intermediate diaphragms and bracing members were modeled discretely in the CSiBridge model, or if their weights were included and distributed to elements or nodes in the model. The weight of those diaphragms and bracings account for approximately two percent of the superstructure weight.
- The load ratings appropriately considered the interaction between axial load, moment, and shear.
- No section loss was considered in the ratings of the rigid frame girders, which is consistent with the observations in the 2013 bridge inspection report.
- The load rating report only provided rating calculations for the rigid frame girder at four locations:
  - At Abutment 1, where shear would be assumed to control
  - The point of maximum positive moment in Span 1
  - Mid-span in Span 2, and
  - End of Span 2, over Bent 2

  It is unknown if rating calculations were performed at other sections of the main girders.
- The load rating summary demonstrated that the operating ratings for the main girders were well above legal weight limits. The minimum calculated rating factor was 2.32, for the TK527 live load model at mid-span of Span 2.

Load rating of the rigid frame legs

- The load rating considered section loss in modeling the lower half of the rigid frame legs. In the webs, this loss was based on what was described as the worst-case observation, an 11" by 3" hole[28]. To model the section loss, the load rater calculated an equivalent plate thickness based on the ratio of the original web dimensions (3'-0" by 0.5" thick) to the web depth assuming an 11" void (11" by 0.5" thick) – yielding an equivalent web plate 3'-0" deep by 0.347" thick. While this could be considered an appropriate approximation for the purpose of evaluating global behavior, this method does not allow for the consideration of localized effects (stresses and instability) resulting from the disturbed stress fields around the corrosion holes in the calculation of capacity.

  In the flanges, section loss was modeled similarly. The one-quarter inch section loss on

[28] The 2013 bridge inspection report, on which the load rating was based, describes holes 11-inches by 6-inches (Photo 57) and 11-inches by 11-inches Photo 61) in the rigid frame leg webs. However, since the equivalent section was calculated based on an 11-inch loss applied uniformly along the lower half of the leg's height, the analysis would not have been affected.

the external half of the flanges was modeled as an equivalent one-eighth inch loss of section on the entire width of both flanges.

- Load ratings were calculated for the frame legs in both bents at sections 47 feet, 30 feet, and 14 feet from the bottom of the leg (measured along its length), and at the shoe. The evaluation of Bent 1 added a calculation for a section 18 feet from the bottom of the leg, where the effective section accounting for the section loss was assumed to start.
- In calculating the critical buckling stress of the rigid frame legs in compression, the load rating discounted the restraint provided by the original rigid cross bracing. This was appropriate since the cross bracing was no longer securely connected to the legs, and in the case of Bent 1, partially removed. However, the calculation of the leg buckling stress, F_{cr}, was based on an equivalent length factor, k, equal to 0.75 for both in-plane (along the strong axis) and out-of-plane (along the weak axis). This was not appropriate as it assumes a much higher level of restraint at the ends of the frame legs than could be provided by the structure, especially in the bridge-transverse direction. As the effective unbraced length of a column greatly affects its ability to resist global buckling, which typically controls its capacity against axial compression, underestimating the value of k could result in an unconservative overestimation of column's capacity.

  The calculations referenced Article 10.54.1.2(a) of the AASHTO *Standard Specifications for Highway Bridges*, 17th Edition for the selection of this value, which prescribes an effective length factor of 0.75 for members with riveted, bolted, or welded end connections that have lateral support in both directions at its ends. The structure as originally constructed, with effective cross bracing, could be considered to be laterally supported. However, the retrofit and removal of the lower bracing in Bent 1, and the lack of stiffness of the severed upper bracing members due to corrosion, changed the structural system. The cable retrofit installed in 2008 likely provided some measure of restraint, but the marginal axial stiffness (and lack of rotational restraint) provided by the cables could not be considered to provide sufficient support for the frames to be considered fully restrained against sidesway.

  Lacking full lateral support, the load rater would have been directed by the Standard Specifications to determine an appropriate effective length factor "by a rational procedure" (Article 10.54.1.2(b)). While not included in the AASHTO Standard Specifications, theoretical values of k considering various levels of restraint are available in other design codes, including the AASHTO *LRFD Bridge Design Specifications* (Figure 24).

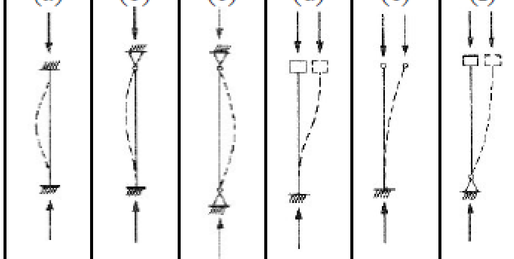
	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Design value of K when ideal conditions are approximated	0.65	0.80	1.0	1.2	2.1	2.0
End condition code		Rotation fixed Rotation free Rotation fixed Rotation free		Translation fixed Translation fixed Translation free Translation free		

Figure 24: Effective Length Factors (AASHTO LRFD Bridge Design Specifications Table C4.6.3.5.1)

- The shoes at the base of the column legs provided restraint against translation in both the bridge-longitudinal and bridge-transverse directions, but no restraint against rotation. In the bridge-longitudinal direction (the strong axis of the frame legs), the bolted connection between the frame leg and the frame girder provided sufficient restraint to be essentially fixed against translation and rotation (column (b) in Figure 24). In the bridge-transverse direction, though, the shallower connection (2 feet with 6 lines of bolts, versus just over 9 feet and 25 lines of bolts in the bridge-longitudinal direction) and relatively flexible cable bracing wouldn't provide sufficient restraint to be considered fixed against translation nor rotation (resulting in a system closer to that in column (f) in Figure 24) and $k = 2.0$ would have been most appropriate for the load rating. With proper consideration of the nominal amount of restraint provided by the cable retrofit, as discussed below in the *Load Effects – Capacity of the Frame Legs* section, this value could be reduced to the range of 1.2 to 1.4.
- The load ratings appropriately considered the interaction between axial load, moment, and shear in the frame legs. However, the use of effective sections to account for section loss did not account for local effects that could affect capacity.
- The shear capacity of the section 18 feet up from the bottom of the leg (measured along its length) in Bent 1 controlled the ratings of the legs, and for the bridge, with operating rating factors of 0.92 and 0.88 for the HS-20 and TK527 trucks, respectively. As discussed above, this is equivalent to vehicle weights of 33 Tons and 35 Tons for the HS-20 and TK527, respectively, and a controlling Safe Load Capacity of 26 Tons, which became the posting weight for the bridge.

- The 2021 routine inspection report indicated that the section loss considered in the 2014 load rating had advanced, with the web portion of the legs exhibiting areas of thick laminar corrosion with "*holes up to 12" L by 12*" H above the bearing stiffeners". It is unclear whether the bearing stiffeners referenced are the same as those labeled as such in Figure 5. Regardless, this change of condition should have triggered a review of the load rating because the 12" reported width of the corrosion was larger than the 11" assumed in the 2014 load rating.

FHWA Load Rating Analysis

The FHWA performed independent structural analyses to verify the applied loads, load effects, and capacities used in the original design and the 2014 load rating. The effects of the additional asphalt wearing surface, effective length factor, and tension tie section loss on those analysis results was also explored.

Applied Loads - Weight of the structure

FHWA compared the dead loads calculated in design, in the 2014 load rating, and in-service at the time of the collapse:

- Sheet 12 of the 1970 design drawings provide notes describing the dead loads considered in the design and stress tables for the rigid frame from which applied dead loads can be extracted.

  The design included an allowance of 250 pounds per lineal foot (plf) per frame to account for future additions. As no additions were made, that load was excluded from FHWA's analysis.

  From the reactions provided in the 1970 stress tables, FHWA calculated that the nominal dead load weight of the bridge used in the original design (including all components above the shoes and the 3-inch wearing surface) was 5,322,000 pounds.

  - Excluding the 250 plf future additions allowance, the nominal weight of the bridge as designed was 5,100,000 pounds.
  - Excluding the weight of the 3" wearing surface, the nominal weight of the bridge was 4,326,000 pounds.
- From the output of the CSiBridge model presented in the 2014 load rating and the bridge geometry, FHWA backcalculated that the dead load weight of the bridge used in that evaluation, including the rigid frame, floor beams, stringers, deck, sidewalk, and 3" wearing surface was 4,975,000 pounds, approximately 2.5 percent lower than the nominal weight calculated for the original design.
- FHWA independently calculated the nominal weight of the bridge to compare dead loads used in the original design and 2014 load ratings with the configuration of the bridge at the time of the collapse.
  - Excluding the weight of the wearing surface, FHWA calculated the total weight of the girders, floor beams, stringers, deck, sidewalk, railings, diaphragms, and bracings as 4,410,000 pounds, approximately 2 percent higher than the nominal

weight calculated for the original design. The presentation of dead load values in the 2014 load rating didn't allow for a similar comparison.

- Including the weight of the asphalt wearing surface measured after the collapse (which averaged 5.6" thick), FHWA calculated the total nominal weight to be 5,831,000 pounds. This is approximately 14.3 percent higher than the dead load of the bridge as designed and 17.2 percent higher than the dead load used in the 2014 load rating.

Load Effects – Axial Forces, Moments, and Shears in the Rigid Frames

As an indeterminate structure, the determination of axial forces, moments, and shears at a given section of a rigid frame will be highly dependent on the distribution of member stiffness in the structural model. Similarly, the global orientation of the frame, on a downward slope, results in a distribution of forces different from that of a level frame that must be accounted for in the structural model. FHWA found that, for any given set of applied loads, there was general agreement between the force effects in the rigid frame between those reported in the original design, the 2014 load rating, and FHWA's structural model.

Load Effects - Tension Stress in the Frame Legs

As described above in the *Fracture Critical Member Procedures* section, FHWA calculated stresses in the frame legs under dead and live load to determine whether the frame legs, or any portion of the frame legs, experienced tension. FHWA's analysis found that the bottom flange of the upper portion of the frame legs would experience up to 7,000 psi of tension under dead and live loads. Also see Figure 11.

Capacity - Moments and Shears in the Frame Girders

FHWA calculated moment and shear capacities for the rigid frame girders and compared the resulting rating factors with those reported in the 2014 load rating. The following was observed:

- FHWA calculated a minimum operating rating factor for the HS-20 live load model, considering the additional dead load from the measured asphalt thickness, of 1.31 for moment in Span 2, and 1.25 for shear near Abutment 1. In the 2014 load rating, the controlling operating rating factors for the HS-20 live load model were 2.72 for moment in Span 1 and 3.16 for shear near Abutment 1. In general, while additional weight from the additional asphalt wearing surface reduced the calculated rating factors for the frame girders, the girders had sufficient capacity to carry typical traffic loads.

Capacity – Axial Loads, Moments, and Shears in the Frame Legs

FHWA analyzed the capacity of the frame legs considering the aforementioned underestimation of the effective length factor, k, and the thickness of the asphalt wearing surface at the time of the collapse. The following were observed:

- FHWA calculated the effect of elastic restraint from the cables and the transverse stiffness of the main girders on the value of k. This analysis established a range of possible values of k as a function of the tension in the cable bracing installed in 2008 and the lateral stiffness of the frame girders and bridge deck (Figure 25). As there was no verification of the tension in the cable bracing other than that they were tight, FHWA assumed that the tension force would be nearer the lower bound of the values in Figure 25 and evaluated the legs assuming a value of k in the weak axis of the leg in the range of 1.2 to 1.4. These values of k reduce the axial compression capacity of the frame legs, based on global buckling, by 45 to 65 percent from that used in the 2014 load rating.

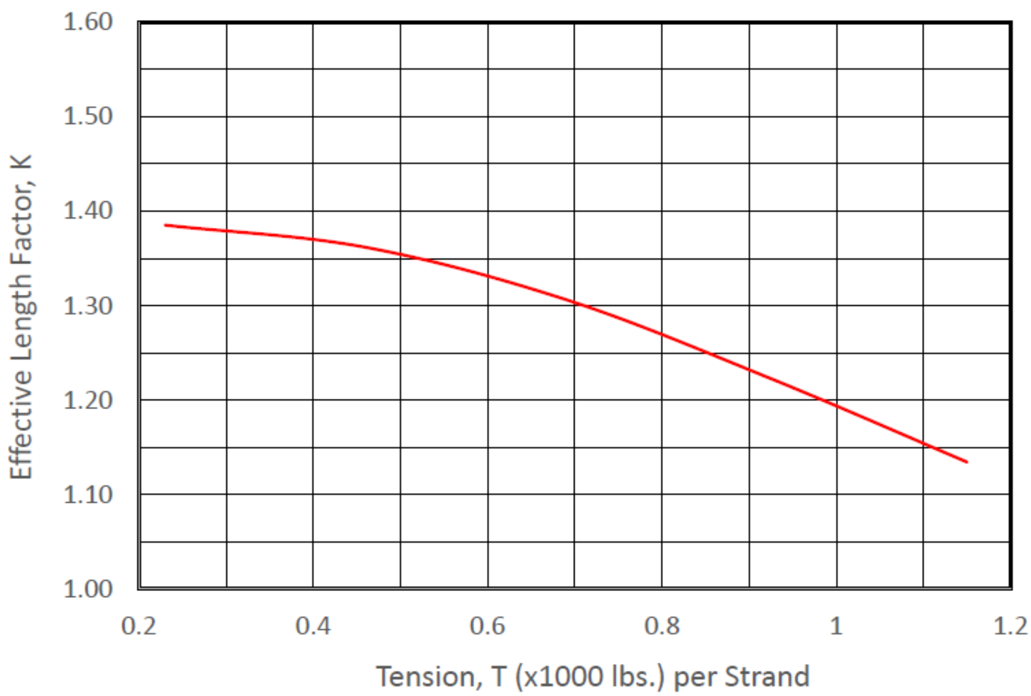


Figure 25: Effective length factor, *k*, of the frame legs versus bracing cable tension and girder/deck lateral stiffness

- FHWA independently calculated capacities of the rigid frame legs under axial compression, flexural moment, and shear using the provisions of the AASHTO *Standard Specifications for Highway Bridges, 17th Edition*. With the exception of the selection of effective length factor and its effect on axial load capacity, FHWA's capacity calculations were in agreement with those made in the 2014 load rating analysis.
- FHWA calculated operating rating factors for the frame legs to determine how the rating would be affected by the two analysis elements (asphalt wearing surface thickness and effective length factor) where FHWA's analysis was in disagreement with the 2014 load rating. The minimum operating rating factors calculated for these cases are presented in Table 5.

Table 5: Minimum calculated operating rating factors

Wearing Surface Thickness	Effective Length Factor, ***k***	Minimum Operating Rating Factor (HS-20 Loading)	Controlling force effect
3”	0.75	0.92[a]	Shear
5.6”	0.75	0.77	Shear
3”	1.2	0.62	Axial load and flexure
5.6”	1.2	0.17	Axial load and flexure
3”	1.4	-0.17	Axial load and flexure
5.6”	1.4	-0.66	Axial load and flexure

[a]This is the value reported in the 2014 load rating and the basis for the 26 ton posting.

- A negative rating factor value indicates that the applied dead load exceeds the bridge’s capacity and no capacity is available to carry vehicular live load. However, as the load rating equation includes factors for uncertainty in applied loads and the estimation of capacity, it is not intended to act as a forensic tool nor indicate the possible mode of failure. Instead, a negative rating factor would have indicated to the load rater and bridge owner that immediate action (bridge closure and/or repair) was required to safeguard the traveling public. The NBIS[29] and MBE requires that bridges not capable of carrying a minimum gross live load of three tons must be closed. For the HS-20 load model, with a gross vehicle weight of 36 tons, this would be the equivalent of a rating factor equal to 0.08.

FHWA also analyzed how consideration of local effects around the holes in the frame leg webs and loss of section in the tension tie plate would have affected the load rating.

- The loss of lateral support from the web plate would result in an unbraced length of the frame leg flange. The bucking stress of this unbraced section would depend on the fixity of a plate and the length assumed to be unbraced and could be less than the stress required to buckle the column globally. The critical global buckling stress of the frame leg described above, with the upper bound value of k, was 11,500 psi. FHWA’s analysis found that the unbraced length of flange would need to be at least 8 feet for this mode of failure to control the analysis. Since the holes in the web plate were on the order of 1 foot, this was not a controlling limit state.
- Using the geometry of the foot and lowest panel of the frame leg, FHWA constructed a strut and tie analytical model to establish a load path through deteriorated section to the shoe, determine loads in the elements making up that load path, and evaluate those elements considering loss of section. Figure 26 shows the geometry of the strut-and-tie model and direction (tension or compression) of the calculated forces in the leg. The model confirmed that the change in direction of the leg flange does introduce enough tension into the tie plate to overcome the compression force in that element due to shear.

[29] 23 CFR 650.313(m)

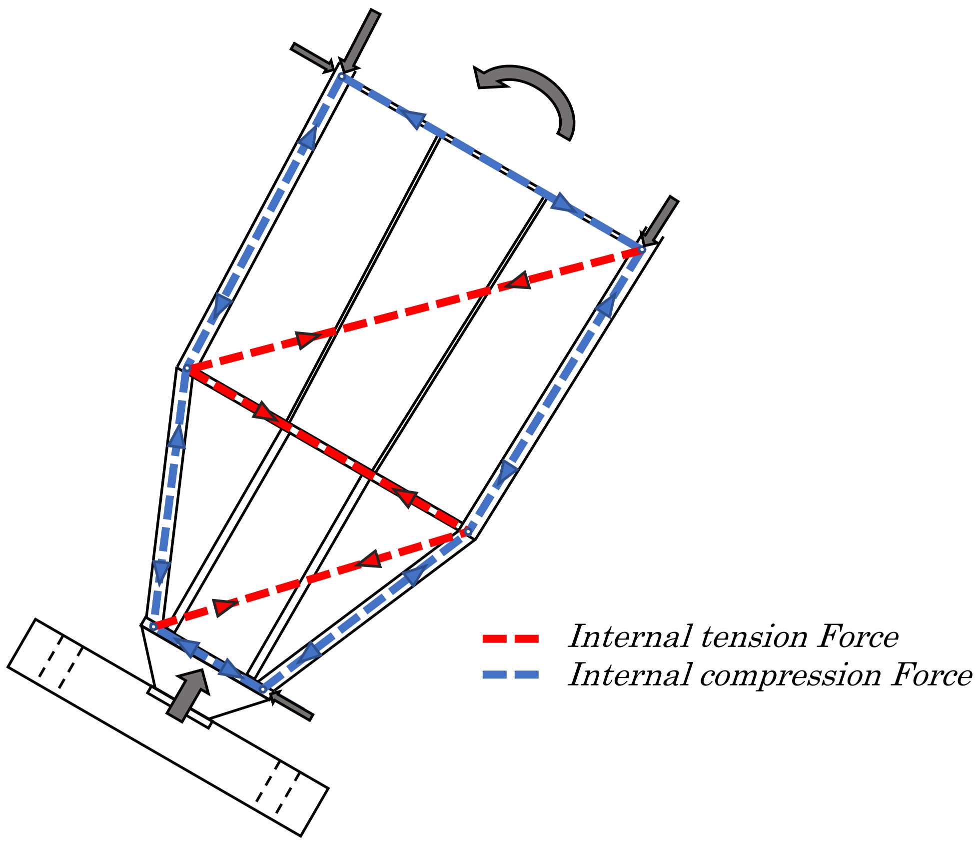


Figure 26: Strut and tie model of the frame leg base and shoe

FHWA's evaluation using the strut-and-tie model determined that, as-designed, the 23- ½" by ¾" (17.63 in^2 total area) tension tie plate had sufficient capacity to safely carry legal loads (operating rating factor equal to 8.8), even with the asphalt wearing surface thickness measured after collapse (reducing the operating rating factor to 8.1). Lack of accurate section measurements prior to collapse precluded FHWA from calculating a load rating based on the conditions at collapse. However, FHWA could determine at which point a rating based on strut-and-tie evaluation of measured section loss in the tension tie would have produced a rating factor lower than those shown in Table 5 above:

Table 6: Tension tie areas returning equivalent operating ratings to sectional analysis

Wearing Surface Thickness	Effective Length Factor, *k*	Minimum Operating Rating Factor from Sectional Analysis (HS-20 Loading)	Equivalent Tension Tie Remaining Section Area
3"	0.75	0.92	6.97 in^2
5.6"	0.75	0.77	7.54 in^2
3"	1.2	0.62	6.56 in^2
5.6"	1.2	0.17	6.71in^2
3"	1.4	-0.17	N/A
5.6"	1.4	-0.66	N/A

- Corrosion mapping performed by NTSB at the FHWA Turner-Fairbank Highway Research Center after the collapse indicated that as little as 2.2 in^2 (out of 17.7 in^2) of steel section remained in the tension tie plate of the Bent 1, right, leg. Thus, per Table 6, if the tie plate had been properly accounted for in the load rating, it would have controlled the rating and possibly resulted in closure of the bridge.
- The strut-and-tie model demonstrates that the through holes observed in the lower corners of the web panel above the shoe were likely in the compression field that would form orthogonal to the diagonal tension tie shown in Figure 26. As these stresses would spread uniformly across the panel, their effect on capacity would be lower, compared to discontinuities and deterioration in areas of concentrated tension stress near the tension tie-to-flange weld.

SUMMARY OF FINDINGS

- Load ratings performed prior to the collapse, and by FHWA after the collapse, indicate that the Fern Hollow Bridge had sufficient capacity, as represented in the design documents, to carry typical highway loads.
- Bridge inspections did not accurately record the thickness of the asphalt wearing surface on the bridge, nor was there coordination between street paving operations and the bridge owner to ensure that wearing surface thicknesses did not exceed those assumed in design and specified in the design documents. As a result, measurements taken after the collapse indicated that the asphalt wearing surface averaged almost two times the thickness assumed in design. This additional load was not accounted for in the 2014 load rating, nor did bridge inspectors identify the additional thickness and require a re-rating of the bridge.
- The use of average reduced section properties in the 2014 load rating to account for section loss in the rigid frame legs, while appropriate for the evaluation of global stability, did not account for local effects due to the through holes in the web that could reduce capacity. The 2014 load rating did not include additional analysis to determine the effects of localized loss on the capacity of the rigid frame legs.
- The 2014 load rating, performed to estimate remaining live load capacity considering deterioration observed during inspection, overestimated the axial load capacity of the rigid frame legs. This miscalculation resulted from an underestimation of the value of the effective length factor, k, used in the calculation of column capacity. The selection of effective length factor appeared to be based on the misapplication of design specification provisions to actual structural conditions that were inconsistent with standard design assumptions.
- Bridge inspections did not provide sufficient data for the load rating engineer to identify the shoe tension tie plate as an element requiring re-evaluation due to deterioration, nor did review of the inspection reports by the load rating engineer identify the tension tie plate as an element requiring re-evaluation.

- Correct calculation of the dead load (due to the thicker wearing surface), axial load capacity of the frame legs using an appropriate effective length factor, or evaluation of the tension capacity of the shoe tension tie plate could have identified the bridge’s lack of capacity prior to collapse.

REFERENCES

1. PennDOT. (2010). *Bridge Safety Inspection Manual Publication 238, 2nd Edition*. Pennsylvania Department of Transportation. Harrisburg, PA..

2. PennDOT. (2022b). *Bridge Management System 2 (BMS2) Coding Manual Publication 100A, Jun. 30, 2022 Edition*. Pennsylvania Department of Transportation. Harrisburg, PA. Last accessed Mar. 16, 2023 https://www.dot.state.pa.us/public/Pubsforms/Publications/PUB%20100A.pdf.

3. FHWA. (1995). *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*. Report No. FHWA-PD-96-001. Federal Highway Administration (FHWA). Washington, D.C. Last accessed Mar. 16, 2023 https://www.fhwa.dot.gov/bridge/mtguide.pdf.

4. FHWA. (2015). *Bridge Inspector's Reference Manual*. Report No. FHWA-NHI-16-013. Federal Highway Administration (FHWA). Vienna, VA.

5. AASHTO. (2008). *The Manual for Bridge Evaluation – 1st Edition.* American Association of State Highway and Transportation Officials (AASHTO). Washington, D.C.

6. AASHTO. (2018). *The Manual for Bridge Evaluation – 3rd Edition*. American Association of State Highway and Transportation Officials (AASHTO). Washington, D.C.

7. AASHTO. (2019). *Manual for Bridge Element Inspection – 2nd Edition*. American Association of State Highway and Transportation Officials (AASHTO). Washington, D.C.

8. eCFR. (2004). *National Bridge Inspection Standards*. 23 CFR §650C. Washington D.C. Last accessed Mar. 16, 2023 https://www.ecfr.gov/on/2022-01-27/title-23/chapter-I/subchapter-G/part-650/subpart-C.

9. FHWA. (2016). *Fracture Critical Inspection Techniques for Steel Bridges – Instructor Guide*. Report No. FHWA-NHI-10-014. Federal Highway Administration (FHWA). Arlington, VA.

APPENDIX A

The following is a review of the inspection notes and sketches, focusing on frame leg and cross bracing condition.

Cross-bracing Condition (from inspection reports):

- September 2005: September 24, 2004 field notes updated with any changes in condition. Bottom lateral bracing member has heavy laminar corrosion with total section loss around 60% of the perimeter at the mid height leg connection. (Bent 1R). Condition mainly generalized with no sketches.
- September 2007: Section loss continues with holes identified. Steel tube lateral wind bracing from girders to midspan floor beams in poor condition. Areas of severe corrosion below weep holes with several missing connection bolts. Cross-bracing at B1R leg has a 3.5"H x 1"W hole on the vertical face of web. Diagonal frame leg−lateral bracing member has a 9" x 6" hole on the top horizontal flange at the B1R. Lateral bracing at Near frame legs with moderate corrosion on the underside of the bottom flange.
- September 2009: Diagonal cross-bracing (welded box section) at near frame (B1), top right connection to right leg has a 2' length of holes rusted thin and through on the vertical face of web. Underside of diagonal bracing at mid−height right leg has a 2' length rusted thin and through. Lower right leg at base has a 2' length of section loss to diagonal brace and gusset plate connection. Left leg, diagonal brace at center connection has 3 of 4 sides 90% rusted through, 50% head loss to the nuts on gusset plate connection, plate is rusted thin and through.

  Both legs were retrofitted with wire cable since the 2007 inspection, plans of the rehabilitation are included. The (2) wire cables attached to each frame are loose and move freely and lack tension. The City was notified of this condition following the inspection.
- September 2011: Cross-bracing – 50% loss of section; cables loose. Note was repeated from September 2009 inspection.
- September 2013: Note was repeated from September 2009 inspection with updated dimensions. Diagonal bracing (welded box section) at Bent 1, top-right connection to right leg has a **2.5** ft. length of holes rusted thin and through on the vertical face of web. Underside of X-bracing at mid-height right leg has a **2.5 ft.** length rusted thin and through. Lower right leg at base has a **2.5 ft**. length of section loss to diagonal brace and gusset plate connection. Left leg, diagonal brace at center connection has 3 of 4 sides 90% rusted through, 50% head loss to the nuts on gusset plate connection, plate is rusted thin and through.
- September 2014: Note was repeated from September 2013 inspection.
  Galvanized steel tensioning cable retrofits at both frame legs were previously reported as being loose. “The wire cables were tightened in the summer of 2014. The cables are tight and in good condition.”
- September 2016: The cross-bracing on the frame legs has a number of large holes, and the connections to the frame legs exhibit 100% section loss. Corrosion continues to progress.

- September 2017: The lower cross-brace on Bent 1 is nearly severed. This member should be reattached as it continues to deteriorate and could fall.
- March 2018: The Bent 1 cross-frame brace at the mid-height connection to the right frame leg is severed. The lower cross- brace connection on Bent 1 is nearly severed.
- September 2018: The Bent 1 cross bracing is in Critical condition. Both of the lower bracing members at the right frame leg are completely severed. The bracing members are resting against the retrofit reinforcing cables. Other bracing members continue to deteriorate at an accelerated rate.
- September 2019: The Bent 1, mid-height and bottom cross-brace connections failed in 2018, and the lower cross-brace was removed. Overall, the frame legs and cross-bracing are deteriorating at an accelerated rate, due to malfunctioning drainage systems and deterioration, contamination and seepage through the deck concrete (previously noted). Cable retrofits added to both bents are in good condition, and are tight.
- September 2020: Note was repeated from the September 2019 inspection.
- September 2021: Note was repeated from the September 2019 inspection.

Frame Leg Condition (from inspection reports):

- September 2005: September 24, 2003 field notes updated with any changes in condition. The frame leg areas located at and below the mid - height lateral bracing connection plates all contain large amounts of rust accumulating in the valleys formed by intersecting stiffeners. The accumulations typically are moist and generating severe corrosion and scaling of the surrounding structural components. The transverse stiffeners attached to the near right (B1R) frame leg exhibit several areas of up to 2" by 4" of 100% section loss and knife edging of the stiffeners. Other transverse stiffeners on all frame legs exhibit similar severe corrosion with up to 75% section loss and some knife edging. The west frame (B1) legs exhibit up to 1/32" loss to the bottom flange and web plates while the east frame (B2) legs exhibit less than 1/32" loss to the bottom flange and web plates.
- September 2007: Notes about the frame legs were listed under substructure portion of the inspection report, although notes about the leg cross-braces were included under the superstructure portion of the inspection report. It is unclear in the 2007 report whether the frame legs were considered in the superstructure or substructure component condition rating; however, it is clear in later reports that the legs were assessed as a part of the superstructure.
  Near (B1) Frame Legs: Frame leg conditions are identical to those discussed earlier. Severe corrosion of column bottom flange up to ⅛" section loss. Web contains areas of severe corrosion with 100% section loss. Transverse stiffeners exhibit knife edging and section loss up to 100%.
  Far (B2) Frame Legs: No condition info mentioned in report narrative but inspection notes carried over from previous inspection and updated regarding percentage of section loss.
- September 2009: Flanges of the lower 25' portion of the frame legs supporting the girders are rusted with areas of thick laminar corrosion and minor section loss. The secondary bracing and stiffening members exhibit areas of severe corrosion and up to ⅛" loss with

deep pitting and small holes through the web and stiffeners; 3 of the 4 legs have transverse stiffeners that are completely rusted through. Both legs were retrofitted with wire cable since the 2007 inspection, plans of the rehabilitation are included in this report. The (2) wire cables attached to each frame are loose and move freely and lack tension. The City was notified of this condition following the inspection.

1. Near (B1) Frame legs; thick laminar corrosion with up to ⅛" section loss of the bottom flange was observed. Web has areas of similar corrosion with 100% section loss. Lower six transverse stiffeners of the right leg exhibit knife edge loss with holes through the stiffeners, (2) additional holes up to 2-1/2" dia. were found in the web at the same location. Lower portion of the left leg has (3) holes through the web plate, up to 1" diameter.
2. Far (B2) Frame legs; web, flange, and stiffener conditions are similar to those of the Near frame. Lower right leg has (2) holes with perforations in the web up to 3" diameter. Right leg has (6) transverse stiffeners rusted thin and through with holes up to 3-1/2"L x 1-1/2"W. Left leg has thick laminar corrosion to lower portion of web, up to 3/16"; transverse stiffeners have 3/16" remaining of original 1 /2".

- September 2011: Flanges of the lower 25' portion of the frame legs supporting the girders are rusted with areas of thick laminar corrosion and minor section loss. The web portion of the legs exhibit areas of thick laminar corrosion with holes up to 3 inches in diameter just above the bearing stiffeners. The secondary bracing and stiffening members exhibit areas of severe corrosion and up to ⅛" loss with deep pitting and small holes through the web and stiffeners; 3 of the 4 legs have transverse stiffeners that are completely rusted through. Both legs were retrofitted with wire cable since the 2007 inspection, plans of the rehabilitation are included in this report. The (2) wire cables attached to each frame are loose and move freely, lacking tension.
- September 2013: Flanges of the lower 25' portion of the frame legs supporting the girders are rusted with areas of thick laminar corrosion and minor section loss. The web portion of the legs exhibit areas of thick laminar corrosion with holes up to **8 inches long by 3 inches high** just above the bearing stiffeners. The secondary bracing and stiffening members exhibit areas of severe corrosion and up to 3/16" loss with deep pitting and small holes through the web and stiffeners all 4 legs have transverse stiffeners that are completely rusted through. Both legs retrofitted with wire cable which are still loose.
    - Near (B1) Frame legs; severe corrosion of bottom flange with up to **3/16"** section loss. Web has areas of severe corrosion with 100% section loss. Lower six transverse stiffeners of the right leg exhibits knife edge loss with holes through the stiffeners, (2) additional holes up to 2-1/2" dia. were found in the web at the same location. Lower portion of the left leg has (3) holes through the web plate, **up to 3" diameter**.
    - Far (B2) Frame legs; web, flange, and stiffener conditions are similar to those of the Near frame. Lower right leg has (3) holes located above thrust block; lower at 2 inch dia.; middle at 11 inch by 3 inch; high at 2.5in. Right leg has (6) transverse stiffeners rusted thin and through with holes up to 3-1/2"L x 1-1/2"W. Left leg has thick laminar corrosion to lower portion of web, up to 3/16"; transverse stiffeners have 3/16" remaining of original 1/2"; **a 6 inch H. by 11 inch W. hole in leg above the bearing.**
- September 2014: Note was repeated from September 2013 inspection.

Interim inspection 9-4-2014: deterioration of the weathering steel frame legs has advanced marginally since the last inspection; however, lowering of the current load rating and posting is not required at this time.

- September 2015: The sketches were improved for the 2015 inspection, and many specific measurements of through-holes and section loss previously in the narrative were now documented in the sketches.
  Notes included: Flanges of the lower 25' portion of the frame legs supporting the girders are rusted with areas of thick laminar corrosion and minor section loss. The web portion of the legs exhibit areas of thick laminar corrosion with holes up to **12" long by 12" high** just above the bearing stiffeners. The secondary bracing and stiffening members exhibit areas of severe corrosion and up to 3/16" loss with deep pitting and small holes through the web and stiffeners; all 4 legs have transverse stiffeners that are completely rusted through. The frame legs and cross bracing are in **poor** condition, and exhibit uneven weathering, laminar rust, severe corrosion and holes.
- September 2016: Note was repeated from September 2015 inspection.
- September 2017: Note was repeated from September 2015 inspection.
- March 2018: Note was repeated from September 2015 inspection.
- September 2018: Note was repeated from September 2015 inspection except the modification to the last sentence. The frame legs and cross bracing are in **poor to critical condition**, and exhibit uneven weathering, laminar rust, severe corrosion and holes.
- September 2019: Note was repeated from September 2018 inspection.
- September 2020: Note was repeated from September 2018 inspection.
- September 2021: Key excepts from the report include: *Flanges of the lower 25' portion of the frame legs supporting the girders are rusted with areas of thick laminar corrosion and less than 1/16" section loss. The web portion of the legs exhibit areas of thick laminar corrosion with holes up to 12" L x 12" H above the bearing stiffeners. The secondary bracing and stiffening members exhibit areas of severe corrosion and up to 3/16" loss with deep pitting and holes through the web and stiffeners; all four (4) legs have transverse stiffeners that are completely rusted through.*
  - At Bent 1, Right Side, Mid Height, Far Face, there is 100% of section loss in the web with 3/4"H x 1/2"W steel remaining.
  - At Bent 1, Right Side, Near Face, 5 stiffeners contain 100% section loss up to 10"L x 7"W.
  - At Bent 1, Left Side, Near Face, there is a 6"L x 4"W hole at the bottom of the web, near the gusset plate.
  - At Bent 1, Left Side, Near Face, 5 stiffeners contain 100% section loss up to 10"L x 7"W.
  - At Bent 1, Left Side, Quarter Height, Near Face, there is a 1'-0" L x 10" H area 100% of section loss in the web.
  - At Bent 1, Left Side, Near Face, the top transverse plate is thin to 1/4" thick.
  - At Bent 2, Right Side, Near Face, 3" W area of 100% section is present in the longitudinal stiffeners at the bottom.
  - The report also notes: “The frame legs are in **poor** condition. The frame legs exhibit laminar rust and holes in the webs. There are 5 stiffeners at each bent leg that exhibit 100% section loss.”

- Side-by-side photos from the 2013 and 2021 inspection reports are provided below. Figure A-1and Figure A-2 show the through hole in the web above the shoe in B1R leg which has visibly increased in size in eight years. Figure A-3 and Figure A-4 show a view down the B1R leg showing more extensive section loss on the web stiffeners but also more visible light coming through the through-hole above the shoe. Figure A-5 and Figure A-6 show Bent 1 with emphasis of the lower cross-brace not present in the 2021 inspection.

Figure A-1. Bent 1 right leg, inside face, far side, from the September 2013 inspection.

Figure A-2. Bent 1 right leg, inside face, far side, from the September 2021 report.

Figure A-3. Looking down Bent 1 right leg, inside face, from the September 2013 report.

Figure A-4. Looking down Bent 1 right leg, inside face, from the September 2021 report.

Figure A-5. Bent 1, from the September 2013 report.

Figure A-6. Bent 1, from the September 2021 report