Steel Fatigue: Details, Categories and Cracking

Fatigue cracking is the most common form of structural damage found in steel highway bridges, and the way an engineer responds to a crack depends on understanding why it formed. This course explains load-induced fatigue and the AASHTO LRFD nominal-stress approach with its eight detail categories (A through E′), the Fatigue I and Fatigue II limit states, distortion-induced fatigue in web gaps and connection angles, how cracks are found, and the repair and retrofit methods documented by FHWA. It draws on FHWA's Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges (FHWA-IF-13-020, Dexter and Ocel, 2013) and Chapter 12, Design for Fatigue, of the Steel Bridge Design Handbook (NSBA/AISC, February 2022 edition, updated to the 9th Edition AASHTO LRFD Bridge Design Specifications). Values quoted from AASHTO tables are reproduced as that handbook states them; confirm them against the edition your agency has adopted.

Learning objectives

- Explain load-induced fatigue
- Use detail categories A through E'
- Recognize distortion-induced fatigue
- Select crack repairs

Why fatigue matters for bridge owners

FHWA's Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges opens with a blunt observation: when cracks are discovered in bridge members or connections in service, fatigue is usually the cause. Fatigue is the formation and growth of a crack under cyclic service loads. A truck crossing a span produces a cycle of stress at every detail in the girders, floor beams and bracing; over decades, tens of millions of these cycles accumulate. The stresses involved are far below the yield strength of the steel, which is what makes fatigue easy to underestimate. A detail can carry its static loads with a comfortable margin and still crack under the repetition of ordinary traffic.

The Steel Bridge Design Handbook (Chapter 12) traces the history. Fatigue cracking was observed in railroad equipment more than 140 years ago, when Wöhler's studies of railway axles showed that stress concentrations and sharp angles produced failures well below yield. In North America, cracks in railroad truss hangers and stringer end connection angles led to laboratory work between 1930 and 1960. Fatigue cracks in steel bridges at the road test program of the 1960s became the genesis of the NCHRP fatigue test program that began at Lehigh University in 1968. Before that program, fatigue rules for welded bridge components rested on small specimens and limited data, which the handbook says explains, in part, the relatively large number of fatigue cracking cases in bridges designed before about 1975.

The FHWA manual makes the same point from the owner's side. Improved fatigue design specifications were implemented in 1975, largely as a result of the NCHRP research at Lehigh. Bridges designed before 1975 often included cover-plated beams, web attachments and other details with low fatigue resistance, and many such bridges were built into the late 1970s before the new rules were widely implemented. Weld quality has also improved significantly because of fracture control plan requirements. Many older structures now carry higher traffic volumes and truck weights than they were designed for, increasing both the number of cycles and the stress range at fatigue-prone details.

A second wave of problems comes from a different mechanism. Before 1985, connection plates for diaphragms and cross-frames were commonly not welded to the girder tension flange, out of concern that a transverse weld on the flange was

fracture-prone. That practice left a short, unstiffened strip of web, the web gap, which bends out of plane when adjacent girders deflect differently. The FHWA manual cites an estimate that distortion-induced cracking is the cause of over 90% of all fatigue cracking, and notes, as an example, that 85% of steel bridges in Minnesota were built before 1986. Since 1985 the AASHTO specifications have required positive attachment of connection plates to both flanges, and bridges designed after 1985 show very few distortion problems.

For an inspector, rater or owner, the practical consequence is that the inventory contains a large population of pre-1975 and pre-1985 steel bridges whose details were not designed for the fatigue demands they now experience. Knowing which details crack, why, and what repairs work is a core competency. This course complements BR-I07 (steel inspection) and BR-ST01 (steel I-girder flexure), and leads into BR-F02 on nonredundant steel tension members.

Fatigue versus fracture

Fatigue and fracture are related but different limit states. The FHWA manual defines fracture as rupture in tension or rapid extension of a crack, leading to gross deformation, loss of function or serviceability, or complete separation of the component. Fatigue may or may not lead to fracture, and fracture may or may not be the result of fatigue. The handbook describes crack growth in three regimes: initiation, steady-state propagation and unstable fracture. Because fabricated steel always contains small crack-like discontinuities, initiation can conservatively be ignored for bridges, leaving two practical regimes: stable fatigue cracking and unstable fracture.

A fatigue crack grows a small amount with each load cycle. When it reaches a critical size, it can extend in an unstable way with no increase in stress, either as a low-energy brittle fracture or, if the material is tough enough, as a ductile tear. The critical flaw size is proportional to the fracture toughness of the material, which depends on temperature and loading rate. For bridge steels, toughness is controlled through Charpy V-notch (CVN) requirements in ASTM A709. The FHWA manual notes that minimum Charpy requirements were first included in A709 for bridge steels in 1974, and the fracture control plan for nonredundant members followed in 1975, both in response to the 1967 collapse of the Point Pleasant (Silver) Bridge after fracture of a single eyebar.

The urgency of a crack depends on where it is. The FHWA manual states that it is urgent to assess the potential for fracture whenever there is a crack in any tension element: a tension member, or the tension flange or tension portion of the web of a flexural member. At the same time, the presence of fatigue cracks does not necessarily mean the structure is unsafe. In some redundant structures a fatigue crack may stop propagating on its own as stresses redistribute. Usually, however, an unmitigated fatigue crack will keep growing and eventually cause a fracture.

Redundancy changes the response. The 2013 manual uses the term fracture critical member (FCM) for a member whose fracture could lead to partial or total collapse, and says a fatigue crack in such a member should warrant immediate action, such as closing lanes, posting the bridge for cars only, or closing it until repairs are made. The current federal terminology for these members is nonredundant steel tension member (NSTM), covered in BR-F02. For redundant members, keeping the bridge open may be justified, but a fracture assessment is still needed to show the probability of fracture at service loads is acceptably small.

The manual recommends, in the absence of other established procedures, conducting fracture assessment with fracture mechanics according to British Standard BS 7910, noting there was then no comparable U.S. standard. Assessment needs material toughness, which may come from mill reports, from the assumption of minimum A709 values for bridges designed after 1974 (with care, since design and construction dates can straddle that year), or from testing cores.

Two other mechanisms are outside fatigue but worth recognizing. Constraint-induced fracture is a sudden brittle failure without perceptible crack growth at highly constrained welded intersections; the handbook cites the Hoan Bridge and SR 422 over the Schuylkill River and lists three conditions acting together: high net tensile stress including residual stress, high triaxial constraint, and a crack-like plane roughly perpendicular to the tensile stress. Stress-corrosion cracking is primarily a concern for very high-strength steels and has occurred in prestressing cables, A490 bolts and out-of-specification A514 plate. Neither is addressed by fatigue repairs.

Why stress range governs

The modern fatigue rules rest on an important simplification: for welded steel details, the live-load stress range is the dominant variable. The FHWA manual summarizes what full-scale tests showed. The strength and type of steel have only a negligible effect on the fatigue resistance of a particular detail. The welding process (SMAW, GMAW, FCAW, SAW and ESW) typically does not affect fatigue resistance. And the dead-load or locked-in stress level is not significant; the primary effect of fatigue loading is captured by the live-load stress range.

The reason is residual stress. When a weld cools, it tries to contract, but the surrounding plate restrains it. The weld and a small volume of adjacent plate are left in tension, balanced by compression in the rest of the plate. The handbook notes that tensile residual stresses can reach the yield strength of the material. Since the potential crack sites at weld toes and roots are already at or near yield in tension, the maximum stress at a weld toe from dead load, live load and residual stress combined is always at or beyond the proportional limit. Adding or removing dead load shifts the nominal mean stress but barely changes the true stress range at the crack site. That is why the AASHTO design check compares a stress range to a resistance, rather than a maximum stress to an allowable.

The handbook draws two practical conclusions. Because steel grade does not matter, a single database of tests on many steels from many countries can be used, which has allowed fatigue rules to be simplified and harmonized internationally. And because stress range governs, the designer and evaluator focus on three things: the number of load cycles the detail experiences, the type of detail, and the stress range at the detail.

Flaws are assumed to be present. The handbook describes the kinds of flaws that occur in a fillet-welded detail: partial penetration, lack of fusion, porosity and inclusions, undercut or micro-flaws at the weld toe, and cracking or inclusions around weld repairs, start-stops and arc strikes. Fabrication inspection controls the larger defects, but small discontinuities at the weld toe are typically too small to detect, and their effect is built into the design curves. In a plain welded girder, laboratory cracks usually start at internal porosity, incomplete fusion or trapped slag in the web-to-flange fillet welds. Attachments such as cover plates, gussets and stiffeners introduce a transverse weld toe that forms a line of elevated tension where cracks start from small, sharp discontinuities.

Not every discontinuity matters. A flaw parallel to the stress field, such as the unfused root of a pair of web-to-flange fillet welds, or the interface between bolt lines in a bolted cover plate, does not open under load and does not grow. Orientation relative to the primary stress is therefore one of the first things to establish when a crack-like indication is found.

The handbook adds a caution for inspectors. In laboratory tests, as much as 80% of the fatigue life had been consumed by the time a crack growing from an internal flaw reached the surface and could be observed. A visible crack is usually a late-stage crack.

Mechanical details generally perform better than welded ones. Drilled or sub-punched and reamed holes cause only modest reductions in fatigue life compared with an unaltered rolled shape, and pretensioned high-strength bolts largely mitigate the

effect of the hole through the local compression they introduce. Punched holes are worse because of imperfections at the hole edge. The handbook states that broadly speaking any mechanical detail has a better fatigue life than its equivalent welded detail, which is one reason bolted repairs are favored later in this course.

S-N curves and the eight detail categories

AASHTO uses the nominal stress approach. The nominal stress is the stress near the detail calculated with simple bending and axial formulas, ignoring local stress concentrations. Those concentrations, both the geometric concentration of the connection and the local notch effect at the weld toe, are already reflected in the test data for each detail type, so the designer does not compute them. The FHWA manual describes how the curves are built: results from full-scale tests are plotted as stress range (S) against cycles to failure (N), where failure is commonly defined as a through-thickness crack. Because weld profiles, residual stresses and discontinuities vary widely, the data scatter, and the design curve is set two standard deviations below the mean of the log of cycles, equivalent to about a 97.5 percent survival rate. The handbook describes the same curves as based on a 98 percent confidence limit, a lower bound of resistance.

Details with similar fatigue strength are grouped into categories. The handbook states that AASHTO LRFD defines eight detail categories: A, B, B′, C, C′, D, E and E′. Category A has the highest fatigue resistance and E′ the lowest. The categories are defined in AASHTO LRFD Table 6.6.1.2.3-1 by verbal descriptions and sketches. On a log-log plot each category is a straight sloping line with a slope of −3.0, imposed because the test data fit that slope closely, and beyond a certain number of cycles the line becomes horizontal.

The sloping portion is the finite-life resistance. The handbook gives AASHTO Equation 6.6.1.2.5-2 as $(\Delta F)n = (A/N)^{1/3}$, with N = (365)(75)n(ADTT)SL, where A is the detail category constant, n is the number of stress range cycles per truck passage, and (ADTT)SL is the single-lane average daily truck traffic. The cube root means a small change in stress range produces a large change in life: halving the stress range increases the predicted life by a factor of eight.

The handbook reproduces the detail category constants (AASHTO Table 6.6.1.2.5-1), in ksi^3: A, 250.0×10^8; B, 120.0×10^8; B′, 61.0×10^8; C and C′, 44.0×10^8; D, 22.0×10^8; E, 11.0×10^8; E′, 3.9×10^8.

The horizontal portion is the constant-amplitude fatigue threshold (CAFT), also called the constant-amplitude fatigue limit (CAFL) in the FHWA manual. Below it, no fatigue cracks are expected under constant-amplitude loading. The handbook's values (AASHTO Table 6.6.1.2.5-3) are: A, 24.0 ksi; B, 16.0 ksi; B′, 12.0 ksi; C, 10.0 ksi; C′, 12.0 ksi; D, 7.0 ksi; E, 4.5 ksi; E′, 2.6 ksi. The FHWA manual notes that depending on category, the threshold begins at 2 to 20 million cycles. Note that C′ shares the C constant but has a higher threshold, which is why C′ is treated as a separate category.

Where do common details fall? The sources reviewed for this course identify several. The FHWA manual says a high-strength bolted connection can be considered a Category B detail, and that welded attachments such as fillet-welded doubler plates are most likely Category E or worse. The termination of a fillet-welded cover plate on the I-95 Yellow Mill Pond bridge was a Category E′ detail. The handbook identifies the base metal at the termination of the horizontal weld of a lateral connection plate to a web as Category E, and the FHWA manual identifies longitudinal gusset plate-to-web weld terminations as Category E and connection plate-to-girder welds as Category C. Butt welds with reinforcement not ground flush in a flat plate are associated with the Category C curve. For any real evaluation, classify the detail from the current AASHTO table, including its sketches and thickness- or length-dependent notes.

The handbook's comparison makes the stakes concrete: at a stress range of about 15 ksi, the average fatigue life of a cover-plated rolled beam would be about 1 million cycles, while a welded built-up girder without the cover plate would exceed 8

million. Detail selection is the single most powerful fatigue decision a designer makes.

Fatigue I, Fatigue II and the fatigue truck

AASHTO LRFD Equation 6.6.1.2.2-1, as reproduced in the handbook, sets the fatigue limit state as $\gamma(\Delta f) \le (\Delta F)n$, where γ is the Fatigue I or Fatigue II load factor, (Δf) is the live-load stress range from the fatigue load, and $(\Delta F)n$ is the nominal fatigue resistance. Since the 2010 5th Edition, AASHTO has used two limit states that represent two regimes of behavior.

Fatigue I represents infinite life. The FHWA manual explains that if this limit state is satisfied, the bridge should not crack during its 75-year design life no matter how many stress cycles are applied. The handbook says the Fatigue I live load factor of 1.75 represents the stress range from the heaviest truck that needs to be considered for fatigue, not the absolute heaviest truck. The resistance for Fatigue I is the threshold, and the handbook's reproduction gives $(\Delta F)n = (\Delta F)TH$ for infinite life. Full-scale variable-amplitude tests, the FHWA manual notes, show that if fewer than 0.01 percent of stress ranges exceed the threshold, cracking will not occur.

Fatigue II represents finite life. Fatigue damage accumulates mostly from the vast number of typical trucks rather than the few heavy ones, so the Fatigue II load factor is less than one, 0.80, and it is paired with the sloping finite-life resistance. BR-D01 discusses the history of these load factors.

The load itself is not HL-93. The handbook explains that AASHTO Article 3.6.1.4 applies only the HL-93 design truck with a fixed rear-axle spacing of 30 feet, representing an average axle spacing, and a dynamic load allowance of 15% rather than the 33% used for strength. The FHWA manual adds that the effective fatigue load is 75% of the design truck, a gross weight of 54 kips, and the maximum fatigue load is 150% of the design truck, 108 kips; these follow from the 0.75 and 1.5 load factors in the 2010 edition the manual describes. Under the current factors of 0.80 and 1.75, the equivalent weights are higher. BR-D02 covers the HL-93 load model.

Which check governs depends on traffic. The handbook reproduces AASHTO Table 6.6.1.2.3-2, the 75-year (ADTT)SL above which the infinite-life check governs, assuming one cycle per truck: A, 690; B, 1,120; B′, 1,350; C, 1,680; C′, 975; D, 2,450; E, 4,615; E′, 8,485 trucks per day. These values are relatively low for most categories, so for most details on most routes Fatigue I governs. Where the projected (ADTT)SL is at or below the tabulated value, Fatigue II with the finite-life resistance may be used, except that for details on fracture-critical (nonredundant) tension members the Fatigue I check with infinite-life resistance must be used.

Cycles per truck matter. The handbook reproduces AASHTO Table 6.6.1.2.5-2: simple-span girders, 1.0; continuous girders near interior supports, 1.5, and elsewhere 1.0; cantilever girders, 5.0; orthotropic deck plate connections subjected to wheel load cycling, 5.0; trusses, 1.0; transverse members spaced more than 20 ft, 1.0, and 20 ft or less, 2.0. For other values of n, divide the ADTT values above by n; for design lives other than 75 years, multiply them by 75 divided by the life sought.

Fatigue is only checked where there is net tension. The handbook explains AASHTO Article 6.6.1.2.1: in regions where unfactored permanent loads produce compression, fatigue needs to be checked only if that compressive stress is less than the maximum tensile live-load stress from the Fatigue I combination. If the stress cycle stays compression-to-compression, a crack will not propagate beyond the heat-affected zone. The FHWA manual makes the same point for inspection: elements whose applied stress range stays in compression need not be inspected closely for fatigue cracks, because cracks driven by weld residual stress arrest as they grow into the compressive zone.

A worked example (illustrative numbers)

The following example uses illustrative numbers chosen only to show how the AASHTO equations, as reproduced in the handbook, fit together. It is not a design and does not represent any real bridge.

Situation (illustrative). A simple-span welded plate girder has a transverse stiffener welded to the web and tension flange, a detail the designer has classified as Category C′ from the current AASHTO table. Analysis with the fatigue truck, including the 15% dynamic load allowance and the distribution factor for one lane, gives an unfactored stress range at the stiffener weld toe of 4.0 ksi. The projected 75-year (ADTT)SL is 2,000 trucks per day.

Step 1: which limit state? For C′, the infinite-life (ADTT)SL from the handbook's table is 975 trucks per day for n = 1.0. For a simple span, n = 1.0. Since 2,000 exceeds 975, the Fatigue I infinite-life check governs.

Step 2: Fatigue I check. Factored stress range = 1.75 × 4.0 = 7.0 ksi. The C′ threshold is 12.0 ksi. Since 7.0 is less than 12.0, the detail satisfies the infinite-life check.

Step 3: what if the detail were Category E? Suppose instead the detail at the same location were a Category E attachment. The threshold is 4.5 ksi, and 7.0 exceeds it, so infinite life is not demonstrated. With (ADTT)SL = 2,000, below the E value of 4,615, Fatigue II may be used. $N = 365 \times 75 \times 1.0 \times 2{,}000 = 54{,}750{,}000$ cycles. $(\Delta F)n = (11.0 \times 10^8 / 54.75 \times 10^6)^{1/3} = (20.1)^{1/3} \approx 2.72$ ksi. The Fatigue II factored stress range is 0.80 × 4.0 = 3.2 ksi, which exceeds 2.72 ksi, so the Category E detail fails the finite-life check. The designer would relocate the attachment to a lower-stress region, change to a better detail, or increase the section.

Step 4: sensitivity. Because of the cube relationship, reducing the stress range from 4.0 to 3.4 ksi (15%) would change the factored Fatigue II range to 2.72 ksi, just meeting the resistance. Small changes in stress range move results a long way.

What the example teaches. First, the category matters more than almost anything else; the same stress range that comfortably passes at C′ fails at E. Second, Fatigue I usually governs on busy routes. Third, evaluation of an existing bridge uses the same building blocks, but AASHTO's Manual for Bridge Evaluation provides methods to estimate minimum, mean and evaluation fatigue life and to account for traffic growth, as the FHWA manual notes; BR-L01 introduces the evaluation framework. Fourth, analysis assumptions matter: the FHWA manual observes that the maximum fatigue loading implied by AASHTO was intentionally calibrated to match earlier specifications, in part because line-girder distribution factors are conservative, and recommends a more realistic maximum truck if refined analysis is used for evaluating cracks or repairs.

In practice, when a crack is being explained or a repair verified, measured stress ranges are usually preferred to calculations. The next sections cover how that is done.

Assessment approaches and field measurement

The FHWA manual describes three analytical approaches for assessing fatigue at a detail, differing in the level of stress refinement. The nominal stress approach, described above, is the simplest and is what AASHTO uses. The local or hot-spot stress approach accounts for the geometric stress concentration in the analysis and uses a single baseline S-N curve. Fracture mechanics models crack growth directly.

The hot-spot approach was developed for offshore tubular structures, where the variety of tube-to-tube connections makes testing every detail impossible. For bridges, the manual says it is useful in understanding cracking from secondary stresses such as web gap distortion, which ordinary bridge analysis does not capture. The manual describes a simple definition of the

hot-spot stress: the stress measured with a 1/8-inch strain gauge placed as close as practical to the weld toe, centered about 1/4 inch from the toe, essentially the AWS D1.1 definition. For assessment of web gap and other bridge details by this approach, the manual recommends using the AASHTO Category C curve with a hot-spot stress range measured or calculated 1/4 inch from the weld toe. It cautions that finite element results for hot-spot stress are highly mesh dependent, and that because the hot-spot approach implies a single cycle count for the threshold, a conservative approach is to ignore the threshold in hot-spot analysis.

Fracture mechanics uses the Paris law, $da/dN = C(\Delta K)^m$, where a is crack size, N is cycles, ΔK is the stress intensity factor range, and the exponent m is 3.0 for carbon steels according to the manual. Integrating the law gives the cycles to grow a crack from one size to another. The manual notes the approach is rarely warranted in design because the initial flaw size is unknown, although an assumed initial semi-elliptical crack depth of about 0.01 inch has given results consistent with test data. It is well suited to an existing visible crack, where the notch effect is no longer significant, and it underlies estimates of how long a crack can safely be monitored.

Field measurement is often the most persuasive tool. The manual describes the infinite-life evaluation with measured data: cracking is explained if the measured maximum stress range exceeds the threshold of the detail, and a repair will be effective if the threshold of the repaired detail exceeds the measured maximum stress range. The maximum stress range is taken at an exceedance probability of 0.01 percent. Measurements must be long enough to capture a statistically meaningful sample of traffic and, where traffic is seasonal, should be made in the season with the heaviest loads. The manual reports that stress range histograms usually converge after two to four weeks of data, sometimes one week. Cycles from cars and pickup trucks are ignored; typically stress ranges below about 0.25 to 0.33 times the threshold are disregarded, which the manual calls a conservative simplification for computing the effective stress range.

Measurement is also how distortion-induced cracking is diagnosed, since the out-of-plane web stresses are rarely predictable from a line-girder model. Strain gauges near a web gap, before and after a trial retrofit such as bolt loosening, can show directly whether the driving stress has been reduced. The manual's chapter on prototype verification describes this practice of measuring the response of a repair before applying it across a bridge.

Finally, remember the variable-amplitude nature of real traffic. The manual notes that Miner's rule can convert a spectrum of variable stress ranges into an effective constant-amplitude stress range that causes the same damage per cycle, which is how measured histograms are compared with the S-N curves. AASHTO's load factors serve the same purpose on the design side.

Distortion-induced fatigue: web gaps and connections

The FHWA manual's chapter on secondary stresses begins with a principle: many design assumptions are simplistic and do not reflect real behavior, and neglecting real behavior gives rise to secondary stresses that can cause fatigue cracking. A simple shear connection with clip angles is assumed not to carry moment, yet a deep connection develops a force couple between its bolts and does transmit moment. Secondary stresses usually develop under displacement control, where two members are constrained to deform together though not intended to do so in design.

The web gap is the classic case. The manual describes the mechanism: when one girder deflects more than its neighbor, it bends a beam-type diaphragm or loads cross-frame members axially, and that force is resisted through a couple by the connection plate welded to the girder web. If the connection plate is not attached to the flange, the force must pass through the short, unstiffened strip of web between the end of the connection plate weld and the web-to-flange weld. In the negative moment region the tension flange is embedded in the concrete deck and cannot move laterally, so the web gap bends sharply in double curvature with each truck passage. The handbook adds that the magnitude of this movement depends on girder

spacing, skew and the type of diaphragm or cross-frame; the FHWA manual notes skewed and curved bridges are more prone because of greater differential deflection and larger cross-frame forces.

The typical crack is horseshoe-shaped, starting at the bottom of the web gap and running down both sides of the connection plate into the web, or a longitudinal crack along the toe of the web-to-flange weld. The handbook observes that, fortunately, most distortion cracks lie in planes parallel to the primary stresses and that their stress intensity may decrease as they lengthen, so they can slow or stop once flexibility is provided. As a result, distortion-induced cracks have not caused significant numbers of fractured flanges. The danger is a crack that turns, after long growth, to run perpendicular to the primary bending stress in the web.

The handbook catalogs other distortion cases: transverse stiffeners cut short of the tension flange, where lateral flange movement during shipping or service can crack the gap in a small number of cycles; floor beam-to-girder connections, where floor beam end rotation is restrained at the top by the deck and forces the girder web to bend at the top of the connection plate; internal diaphragm connection plates in box girders, where box distortion cracks both top and bottom web gaps; and lateral bracing connections. Coped beams, where flame-cut copes with small, rough radii crack at a nominally zero-stress location because the connection actually restrains rotation, are a related case.

Connection angles in stringer and floor beam framing crack for a similar reason. The FHWA manual explains that angles assumed simple have enough flexural rigidity to resist the end rotations imposed by differential displacement, and they crack because they are too thick. It reproduces a published guideline relating angle thickness to bolt gauge and member spacing, and cites research suggesting the maximum measured bending stress range in the angle stay below 24 ksi, the Category A threshold, to avoid cracking.

Current detailing rules address these mechanisms. The handbook explains AASHTO Article 6.6.1.3.1: transverse connection plates must be positively attached to both flanges, and in the absence of better information the connection in straight, non-skewed bridges should be designed for a minimum factored lateral force of 20 kips; for skewed and curved bridges the force can be determined by analysis. A limited exception allows less than full-depth end angles or connection plates on rolled beams in straight bridges with supports skewed not more than 10 degrees and diaphragms in contiguous lines, provided they are at least two-thirds the web depth with a minimum 3-inch gap to each flange.

Finding fatigue cracks

The FHWA manual is candid that crack detection can seem like a black art, especially on painted or rusted steel, and that a keen eye is the best tool. When a crack opens and closes, its faces rub and produce a fine steel powder that oxidizes, bleeding out of the crack as rust staining or discoloration. This can point quickly to problem areas but should not be the only means of detection.

Two nondestructive methods are standard in the field. Dye penetrant is a three-part system in small pressurized cans. The area is cleaned and degreased, with heavy corrosion removed by wire brush; the manual cautions against grinding because it tends to smear the crack closed. A liquid dye, commonly red, is sprayed on and allowed to seep into cracks for a short dwell (the manual cites about 60 to 90 seconds), the excess is wiped off, and a white developer draws the dye back out of the crack so it shows clearly. Magnetic particle testing uses a handheld electromagnetic yoke to induce a magnetic field; a crack disrupts the field and concentrates flux around it, so fine iron powder sprinkled on the surface collects at the crack. The method finds surface cracks and sometimes cracks just below the surface. It can work through paint, but not reliably, and the manual recommends removing paint for best results.

Other methods, including eddy current, manual or automated ultrasonic testing, and time-of-flight diffraction, have been used successfully but cost more and require expensive equipment and highly trained operators. BR-N02 covers these steel NDE methods in more depth.

Where one crack is found, more are likely. Because bridge details repeat, the manual calls the first and most urgent step in planning repairs a special in-depth inspection of the whole bridge for other cracks. It is typically visual but may use magnetic particle, dye penetrant or other methods, especially on nonredundant members. The focus should be tension elements similar to the one that cracked, with priority on details with high live-load stress ranges or on nonredundant members.

Locating the crack tip matters for every repair. The manual's hole drilling procedure starts with identifying the tip using dye penetrant or magnetic particle testing and marking it with a center punch; for through-thickness cracks, it suggests drilling a small reference hole, about 1/8 inch, to check the tip location on the far face. A repair that misses the tip leaves a small crack that will continue to grow.

Documentation completes the job. The manual's repair procedures end by recording the location and repair method in the bridge file for future inspections. BR-I10 covers inspection reports. When cracks are found during inspection, agencies' critical finding procedures may apply, as discussed in BR-I03; a crack in a tension element of a nonredundant member is the type of condition those procedures were designed to escalate.

Inspectors should also know the benign cases. The manual notes that temporary tack welds on older structures, used to hold parts during riveting or bolting, often crack. Usually cracked and uncracked tack welds are benign. The concern arises if a crack has propagated into the base metal, or for tack welds aligned parallel to the primary stress, which the manual classifies as Category E. Tack welds that have cracked through their throat and are less than 2 inches long in the stress direction can usually be evaluated as Category C.

Choosing a strategy: monitor, repair or retrofit

The FHWA manual distinguishes repair from retrofit. A repair is intended to arrest propagation of an existing fatigue crack; a retrofit is intended to upgrade fatigue resistance and prevent future cracking. Retrofits are usually applied at details similar to those that have already cracked. The same treatments are often used for both purposes.

After inspection and any fracture assessment, the type and cause of each crack must be determined. Sometimes that requires removing a drilled core containing the crack so the crack surfaces can be examined. A newly found crack may have existed since fabrication, for example from hydrogen cracking of a weld. The manual says that if a fabrication defect has been exposed to typical service temperatures and loads without propagating by fatigue or causing a fracture, it may be safe to delay repair and monitor it, provided an analysis supports that decision.

For common redundant bridges, the manual states, the best choice is often not to repair a crack but to monitor it, especially for distortion cracks at secondary elements. But monitoring must be more than casual visual inspection; it must use in-depth techniques that reliably locate the crack tips and track growth over time. Even when repair is warranted, it need not be immediate: fracture mechanics can estimate how long a crack will take to reach critical size. In practice, the manual notes, fatigue cracks are usually repaired within months of discovery.

The manual groups repair and retrofit methods into three families: surface treatments of weld toes; repair of through-thickness cracks; and modification of the connection or global structure to reduce the cause of cracking. The choice depends on the circumstances of the cracking and may depend on which skills and tools local contractors have.

The governing question for any repair is whether it removes the cause. For load-induced cracks, the cause is a stress range too high for the detail, so a lasting repair either reduces the stress range or upgrades the detail category. For distortion cracks, the cause is the out-of-plane displacement, and the manual is emphatic that if the driving force is not eliminated, the repair is only temporary. A symmetric approach is needed: if connection plates are on both sides of a web and a softening retrofit is used, both must be treated.

The manual also flags practical constraints. Welding repairs on high-strength steels (yield strength 70 ksi or more) carry a high risk of hydrogen cracking and should only be used if bolted alternatives are absolutely not possible. Welding near flanges embedded in concrete is hard because the deck acts as a heat sink, making preheat and interpass temperatures difficult to maintain. Work in service raises the question of traffic-induced movement during welding; the manual recommends closing lanes over and adjacent to the girder at least for the root pass of a weld that will tie a connection plate to a flange.

Finally, a retrofit must not create a new problem. The manual repeatedly warns that a welded attachment added as a repair can itself be a worse fatigue detail than the one it replaces, and it recommends bolted solutions wherever practical. It also notes the effect of a repair on other elements: for example, if a diaphragm is removed to stop distortion, girder stability and load distribution must be checked.

Repair methods for cracks and weld toes

Surface treatments. The FHWA manual describes grinding, gas tungsten arc (GTA) remelting and impact treatments as weld improvement methods for uncracked welds. They work by improving weld geometry, removing the worst discontinuities, and reducing tensile or introducing compressive residual stress. Some can also repair shallow surface cracks; hammer peening is cited for cracks up to 1/8 inch deep and GTA remelting for cracks up to about 3/16 inch provided the crack is fully removed. Once applied to welds that have been in service, the manual says the remaining life is at least that of the original detail when new, assuming the stress histogram stays about the same, and in most cases the treated detail is at least one category better.

Grinding to remove small edge cracks should be tapered at a 2.5:1 slope with finish grinding parallel to the stress, so an edge crack 1/2 inch deep is faired out over at least 2.5 inches. Weld toe grinding with a disc should go at least 1/32 inch below the toe and no more than 5/64 inch or 5% of the plate thickness. Grinding merely to remove toe microcracks does not appreciably increase fatigue strength.

Air hammer peening uses a blunt tip to plastically deform the weld toe. The manual reports best results with lower air pressure (about 40 psi) and up to six passes, with a recommended indentation depth of about 0.03 to 0.06 inch. Because the benefit comes from compressive residual stress, impact treatments are most effective when applied under dead load, which makes them best suited to retrofits of existing bridges. Ultrasonic impact treatment (UIT) was reported to be more effective than hammer peening but was proprietary when the manual was written.

At the Yellow Mill Pond bridge on I-95, the manual reports, cover plate ends were retrofitted by hammer peening and GTA remelting without closing the road. When girders were later tested at Lehigh after about 20 years and an estimated 56 million truck passages, no cracks were found in the retrofits, and all but one of the treated beams attained Category C resistance compared with E′ for the untreated detail.

Hole drilling. Drilling a hole at the crack tip removes the sharp notch and is perhaps the most widely used repair. The manual says holes of 2 to 4 inches have proven successful, provides an equation for an engineered hole size based on stress range and crack length, requires a minimum diameter of 1 inch, and says the computed diameter should be increased 25% for edge cracks. Holes must remove the tip, be drilled (never flame cut), and be ground smooth at both edges. A fully tensioned high-

strength bolt with hardened washers in a 1- to 1.5-inch hole adds beneficial compression. In distortion cases, the manual cites Lehigh research that hole drilling alone suffices only when in-plane bending stress is below 6 ksi and out-of-plane stress range below 15 ksi, and workshop experience that it will not work alone on skewed bridges. Repeated small holes produce a swiss-cheese pattern of reinitiated cracks.

Vee-and-weld. Gouging out a long crack and replacing it with weld can restore a cracked weld detail to its original life, but the manual says weld repair of base metal where there was no weld is rarely effective and usually recracks sooner unless the stress range is also reduced. Field welds should use low-hydrogen consumables with toughness requirements and be made by welders at least as skilled as shop welders.

Bolted doubler and splice plates. Adding material reduces stress range or restores a cracked section. The manual recommends only bolted doublers, since a slip-critical bolted connection is Category B while a welded doubler is likely E or worse, and requires drilling the crack tip with a minimum 1-inch hole. Bolted splices over cracked cover plate ends tested at Purdue attained Category B resistance.

Post-tensioning can close crack faces and push the stress range into compression; the manual recommends drilling the crack tip as well. **Eyebars and hangers** are a special case: the manual says no repair can be recommended for cracked eyebars, only replacement or added redundancy, with parallel post-tensioning as a temporary secondary load path.

Retrofits for web gap distortion

Because distortion cracks are driven by displacement, the FHWA manual's retrofits for them aim to remove or reduce the out-of-plane bending. They fall into removal, stiffening and softening strategies.

Diaphragm or cross-frame removal. Removing the bracing that causes distortion eliminates the driving force, but only after detailed analysis. The manual reports one Alabama interstate bridge where removing positive moment diaphragms raised bottom flange stress by at most 15%, judged acceptable, and a Texas skewed bridge study where stresses rose 25% and remaining cross-frame forces increased, judged unacceptable. Workshop participants noted that in negative moment regions the bottom compression flange may need the bracing for lateral-torsional buckling, that removed bracing complicates future deck replacement, that diaphragms help distribute truck impact loads, and that records of removal may be lost.

Repositioning. On the approach spans of the I-35W bridge in Minneapolis, the manual describes a University of Minnesota retrofit that removed rivets, lowered the diaphragms as close as possible to the bottom flange, and reconnected them with only four high-strength bolts, since in the negative moment region only the bottom compression flange needs bracing.

Bolt loosening. Measured relative displacements between connection plate and flange are small, roughly 0.001 to 0.030 inch, while bolt holes are larger than the bolts. Iowa State research found web gap strains dropped about 30 to 80% in most cases when bolts were loosened, though K-type cross-frames sometimes showed increases. Results vary with fit-up, so the manual recommends loosening bolts sequentially while measuring strain until the desired reduction is achieved, and securing nuts against backing off.

Stiffening. Positively attaching the connection plate to the flange eliminates or greatly reduces the relative movement. The manual says it is effective for diaphragm and cross-bracing systems but not where floor beams transfer large moments, where softening should be used instead. Welded attachment is the most effective, but added welds have themselves cracked, and the difficulties include high-strength steel, flanges embedded in concrete, and movement under traffic. The manual says the weld should be sized for the 20-kip force of AASHTO C6.6.1.3.1 on right bridges, at least doubled for highly skewed bridges, and should not wrap around the plate or intersect the flange-web weld. Bolted alternatives use a rolled tee (preferred over double

angles for stiffness) with at least four bolts into the flange and four into the connection plate, with legs or stems at least 3/4 inch thick; one Iowa project in which a contractor substituted welded built-up tees saw them crack. Bolting to a flange embedded in concrete may require removing part of the deck or haunch.

Softening. Cutting away part of the connection plate lengthens the web gap and reduces distortion stress. The manual recommends a new gap of one-sixth the web depth but no more than 15 inches, used for example on the Poplar Street Bridge complex in East St. Louis. The remaining stub must be ground flush with the web; on the Lexington Avenue Bridge in St. Paul, incomplete grinding led to quick reinitiation. A faster alternative is a large hole, 3 inches or more in diameter, drilled at the web gap to remove crack tips and soften the gap.

Whatever the method, the manual recommends verifying the response with strain measurements on a prototype before applying the retrofit bridge-wide.

Putting it together: an owner's checklist

The sources reviewed for this course support a consistent sequence for responding to fatigue cracking in an existing steel bridge. The checklist below summarizes it; it is a teaching aid, not an agency procedure.

1. **Characterize the crack.** Locate the tips with dye penetrant or magnetic particle testing, record length, orientation relative to primary stress, and whether it is in a tension element. Photograph and document per BR-I10.
2. **Assess urgency.** Determine whether the member is nonredundant (BR-F02). A crack in a tension element of a nonredundant member warrants immediate action under the FHWA manual's guidance and your agency's critical finding procedure (BR-I03). For redundant members, perform or commission a fracture assessment.
3. **Inspect for similar cracks.** Conduct the special in-depth inspection of similar tension details the manual calls the first and most urgent step.
4. **Determine the cause.** Classify the crack as load-induced (perpendicular to primary stress at a known detail category) or distortion-induced (web gaps, connection plates, angles, copes). Consider fabrication defects. If needed, measure stress ranges in the heavy-traffic season.
5. **Decide: monitor, repair or retrofit.** For distortion cracks in redundant bridges, monitoring with reliable tip location may be the best choice. For load-induced cracks, repair is usually completed within months.
6. **Remove the cause.** Reduce stress range or upgrade the category for load-induced cracks; eliminate or reduce out-of-plane displacement for distortion cracks. Favor bolted over welded solutions.
7. **Verify.** Use strain measurements on a prototype repair where the outcome is uncertain, and confirm the repaired detail's threshold exceeds the measured maximum stress range.
8. **Retrofit similar details.** Extend proven treatments to similar uncracked details, and record everything in the bridge file.

Common pitfalls the sources warn about. Drilling small holes repeatedly at web gaps without addressing distortion. Flame cutting holes. Leaving a softening stub not ground flush. Using welded built-up tees or welded doublers that create Category E details. Weld-repairing base metal without reducing stress range. Removing bracing without checking stability, deck replacement and impact consequences. Treating a single stain or visual indication as a complete inspection.

For designers of new work. The handbook's message is that proper detailing is the main defense against distortion-induced fatigue and that detail selection controls load-induced fatigue. Attach connection plates to both flanges, avoid cover plates and low-category attachments in high-stress tension zones, avoid copes where possible, connect lateral bracing directly to flanges

and preferably by bolting, and avoid constraint-prone intersecting welds. For details on nonredundant tension members, the infinite-life check is required regardless of traffic.

For load raters. Fatigue is evaluated separately from strength load rating. A bridge with an adequate rating factor may still have a limited remaining fatigue life at a poor detail. Where the MBE fatigue evaluation shows limited remaining life, options include more frequent inspection, retrofitting the detail, or refined measurement. BR-L04 discusses how condition affects rating, and BR-ST01 covers the strength design of the same girders.

Above all, use current documents. This course summarizes the FHWA repair manual and the 2022 handbook chapter; detail classifications, thresholds and article numbers should be confirmed against the AASHTO LRFD edition and interims your agency has adopted before any evaluation or design is sealed.

Key takeaways

- Fatigue is usually the cause of cracks found in steel bridges in service; it is driven by live-load stress range, not maximum stress, because weld residual stresses are near yield (FHWA-IF-13-020; SBDH Ch. 12).
- AASHTO groups details into eight categories, A, B, B′, C, C′, D, E and E′, with A best and E′ worst; each has a −3 slope S-N line and a constant-amplitude threshold.
- Fatigue I (load factor 1.75) checks infinite life against the threshold and usually governs on busy routes; Fatigue II (0.80) checks finite life with (ΔF)n = (A/N)^1/3.
- The fatigue load is the HL-93 design truck with a fixed 30-ft rear axle spacing and 15% dynamic load allowance; fatigue is checked only where there is net tension.
- Distortion-induced cracking in web gaps at connection plates not attached to the flange has been estimated to cause over 90% of fatigue cracks; post-1985 details require positive attachment.
- After finding a crack: locate the tips, judge redundancy and urgency, inspect similar details, find the cause, then monitor, repair or retrofit so the cause is removed.
- Drilled crack-tip holes need a minimum 1-inch diameter, must remove the tip and be ground smooth; flame-cut holes should never be used.
- Bolted doublers (Category B) are preferred over welded ones (likely Category E or worse); vee-and-weld rarely works in unwelded base metal unless stress range is reduced.
- Web gap retrofits remove, stiffen or soften the gap; verify with strain measurements, and treat both sides symmetrically.
- Confirm all AASHTO table values and article numbers against your agency's adopted edition.

References

1. FHWA, Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges, FHWA-IF-13-020, 2013
2. NSBA/AISC, Steel Bridge Design Handbook, Chapter 12: Design for Fatigue (D. Mertz; revised by M. A. Grubb), February 2022, prepared to the 9th Edition AASHTO LRFD Bridge Design Specifications; successor to FHWA-HIF-16-002 Vol. 12

Study text only; the final quiz is taken online. Verify all references against the edition your agency has adopted.