

Damage Evaluation and Repair of Prestressed Members

Overheight vehicles strike prestressed concrete girders on overpasses every year, and corrosion attacks strands in girders under leaking joints and in marine air. This course covers how to inspect, assess and repair that damage. It is built on three public sources. The first is NCHRP Report 280, Guidelines for Evaluation and Repair of Prestressed Concrete Bridge Members (Shanafelt and Horn, 1985), which includes full-scale tests on an AASHTO Type III girder. The second is its predecessor, NCHRP Report 226, Damage Evaluation and Repair Methods for Prestressed Concrete Bridge Members (1980). The third is the 2012 Guide to Recommended Practice for the Repair of Impact-Damaged Prestressed Concrete Bridge Girders, from NCHRP Project 20-07 Task 307 (Harries and others, University of Pittsburgh), which adds CFRP methods and a damage classification. Code provisions are quoted as these sources state them; confirm article numbers against current editions. Worked numbers are labeled illustrative.

Learning objectives

- Classify impact and corrosion damage
- Evaluate loss of strands
- Select repair approaches
- Know when replacement is needed

Why damage to prestressed members is different

A reinforced concrete beam that loses some cover or a bar can often be patched and forgotten. A prestressed girder is different, for three reasons that run through every part of this course.

First, the strands are the member. Pretensioned girders carry their dead and live load moment largely because the strands hold the bottom of the section in compression. The strands are concentrated in the bottom flange, at minimum cover, exactly where an overheight truck or a load on a trailer hits. Report 280 notes that because strands sit near the bottom with minimum cover, even scraping off the surface finish may let corrosive elements reach the steel. A few severed strands can remove a meaningful fraction of the prestress at a section. BR-P01 covers how that precompression gets into the girder and how losses are estimated.

Second, concrete that is replaced loses its precompression. A patch placed in the bottom flange of a stressed girder carries no prestress. When live load comes on, the girder bottom goes toward tension, and an unstressed patch or an epoxy-filled crack is the first thing to crack. That is why preloading and active (post-tensioned) repairs appear so often in the literature. A repair that restores area without restoring compression may restore strength but not service performance.

Third, strand damage is partly hidden. Strands may be nicked, gouged, flattened or partly severed under concrete that is only cracked. Shattered concrete may conceal intact strands, or strands that have lost tension. Report 280 describes impacts in which all of the concrete in the bottom flange and web was removed with no apparent damage to the strands, and says that when the bottom flange and part of the web have been shattered for a considerable length, the amount of prestress force left in the strands may be in question.

Causes of damage

Report 280 lists the causes it addresses: vehicle impact, which was by far the most common, plus environmental attack, fire and damage during manufacture, handling or construction. The 2012 Guide focuses on impact damage to girders from overheight vehicles, which typically damages the exterior (fascia) girder, and adds corrosion mitigation for strands that the impact exposes. Corrosion of prestressing steel is considered more serious than corrosion of mild steel; Report 226 makes this point. BR-C02 covers concrete deterioration and repair materials in general; BR-I06 covers inspection of concrete superstructures; BR-N01 covers nondestructive testing that can help locate delaminations and corrosion.

Decisions to be made

Both NCHRP reports frame the problem the same way. After an incident, the agency must decide, in order:

1. Is the bridge safe to carry traffic now, and under what restrictions?
2. Should the damaged member be left alone, repaired in place or replaced?
3. If repaired, which method restores the strength and durability needed?
4. How will the repair be monitored?

Report 226 found that agencies used a range of methods and that practices were not uniform; Report 280 was written to give a consistent set of guidelines, verified by tests. The 2012 Guide updated those guidelines, finding that repair in place had become more common, and that most severe-damage cases still ended with girder replacement.

Repair in place versus replacement

Report 280 emphasizes that repair in place is generally faster, cheaper and less disruptive than replacement, because replacement requires removing at least part of the deck slab, procuring a new girder and closing lanes above and below. Its full-scale tests were intended to show that repair-in-place methods could actually restore strength. External post-tensioning, internal strand splicing and a metal sleeve splice were all tested on a 60-ft AASHTO Type III girder. Later sections of this course work through each method.

Inspecting the damage

Report 280's guidelines start with procedures set up before any damage occurs. It recommends that agencies establish standard procedures for inspection, reporting, engineering assessment and monitoring of completed repairs, "without pressure from an emergency." Because the bridge engineer usually prepares the repair plans, that office should be responsible for inspection, and where organizational constraints put it elsewhere, cooperation with bridge engineering staff should be required. Severe damage should be inspected by competent structural engineers.

Initial inspection and action

The primary objectives of the initial inspection are user safety and preventing further damage. When damage is severe, an experienced structural engineer should make the first inspection and decide whether to restrict traffic or close the bridge. Temporary supports, if required, should be placed immediately. These first actions are normally based on judgment supplemented by brief calculations.

Report 280 offers some reassurance and some warnings:

- Prestressed members normally sustain large vertical deflections without collapsing, and those deflections are readily visible.
- Few bridge closures are necessary for accidental damage to prestressed members.
- If a severely damaged member is fracture critical, meaning its failure would collapse the bridge or span, immediate steps should be taken to prevent collapse.
- When safety is in question, the bridge should stay closed until it is conclusively shown that traffic can be restored.
"Political and public pressure should not be allowed to cause unwarranted decisions when safety is involved."

Sequence and equipment

Inspect the most critically damaged area first, then other damage in descending order of severity; girders first, diaphragms and secondary members last. Record all areas inspected, including those with no damage. Report 280's equipment list includes:

- Good access, such as a platform truck or "cherry picker," after checking that the damaged girder can support the inspection load.
- Good lighting, a carpenter's level to check plumb, and a flashlight and mirror for hidden areas.
- Magnifiers for cracks. With good lighting, a 0.004-in. (0.1 mm) crack can be detected and measured; a 20X microscope can measure cracks to 0.002 in. (0.05 mm).
- String lines for lateral offset and curvature. Relative vertical displacement is best obtained from elevations along girder centerlines at the top of the roadway slab.
- Good cameras, with photos labeled for object, direction of view and approximate distance.

Report 280 also notes that impact is not believed to reduce concrete strength; it cracks, loosens or shatters concrete. Cores can be taken to assess environmental damage, with strand locations considered first. Small spalls and gouges should be measured because reduced cover may affect strand durability.

Strands

The effect on prestressing elements is "of prime importance" in deciding whether to repair or replace. Strands that are severed, nicked, gouged, deformed or sharply bent should be described and located. Concrete can be carefully removed to inspect strands more closely; Report 280 recommends a rivet-gun chipper near strands. Inspection must be tied to assessment, so effort is not spent sampling strands on a girder that will be replaced anyway. BR-I03 covers inspection documentation in general.

The inspection report

Report 280 recommends a standard form, with one sheet per member, and lists its contents. The main items are:

- Bridge identification, location, date of damage and inspection, law enforcement accident report and cause of damage.
- Site conditions (over traffic, over water, other) and user inconvenience.
- Member type, identification and category: fracture critical, primary or secondary.
- Damage to prestressing elements: exposed, nicked, gouged, deformed or severed, with locations.
- Damage to concrete: spalls, nicks, scrapes and gouges with sizes; cracks with width, length and configuration; loose and shattered concrete.
- Member displacement: lateral, vertical, rotation or twist.
- Photographs, factors that may affect the repair, and initial actions such as traffic restriction or strengthening.

One instruction stands out: "The inspection report should not normally contain recommended repair procedures." Facts and judgment are kept separate. Report 280 also recommends a cross-referenced record system covering every damage incident, organized by member type and cause, including the assessment calculations, repair calculations and repair details.

Monitoring repairs

Report 280 treats monitoring as part of inspection, so inspectors can learn from how repairs perform. Follow-up inspection of repairs should be on a regular basis. Experience showed restoration "may not be complete for all damages": patches and repaired cracks placed without preload or post-tensioning may crack and affect durability, and environmental or fire damage carries some doubt. Fully restored members should be inspected at the same frequency as the bridge; repairs with reasonable doubt about strength or durability should be inspected more often, at a frequency set by those responsible for the repair, with the bridge maintenance engineer advised. BR-L04 covers how such findings feed into rating and posting.

Classifying damage

The 2012 Guide gives a damage classification (its Table 1) that ties the condition of the girder to the type of repair. The Guide stresses that classification "must be considered on a case-by-case basis" and that the table gives approximate guidance only.

Class	Description (summarized)	Strand loss	Camber
MINOR	Shallow spalls, nicks and cracks, scrapes, some efflorescence, rust or water stains. No effect on capacity; repairs are aesthetic and preventive.	No exposed strands	No effect
MODERATE	Larger cracks and enough spalling or concrete loss to expose strands. No effect on capacity; repairs prevent further deterioration.	Exposed, none severed	No effect
SEVERE I	Affects capacity but may not be critical, being minor or away from a critical section. Repairs against deterioration warranted; structural repair typically not required.	Less than 5%	Partial loss
SEVERE II	Needs structural repair that can use a non-prestressed method: a STRENGTH limit state repair.	Greater than 5%	Complete loss
SEVERE III	Decompression of the tensile soffit. Needs repair that replaces prestressing force: SERVICE and STRENGTH.	Exceeding 20%; in longer, heavily loaded sections decompression may not occur until close to 30%	Vertical deflection less than 0.5%
SEVERE IV	Too extensive; repair not practical; replace the element.	Greater than 35%	Vertical deflection greater than 0.5%

The "repair or replace" threshold

The line between SEVERE III and SEVERE IV is the repair-or-replace criterion. Beyond Table 1, the Guide lists conditions that require girder replacement:

- Permanent lateral deflection exceeding standard girder tolerance (citing NCHRP 280).
- Permanent vertical deflection from horizontal exceeding 0.5%.
- Cracks at the web/flange interface that remain open, indicating yield of the transverse steel.
- Loss of prestress at a harping point.

- Loss of prestress at girder ends exceeding 25% of the total number of strands (the Guide cites AASHTO LRFD 5.11.4.3).
- Damaged capacity below the residual capacity limit (covered later) if external repairs are used.

The "repair or do nothing" threshold

Between SEVERE I and SEVERE II, the Guide says there are few clear delineators and the decision will generally rest on the engineer's judgment. Often the loss of a few discrete strands in a section will not warrant structural repair, and CFRP techniques have a practical lower limit below which they are not worth using. Whatever the class, the Guide is firm about one thing: wherever possible, strand, "whether severed or in good shape," should not remain exposed to the environment, since deterioration will accelerate and propagate. Even a girder that needs no structural repair needs a sound patch.

Girder type matters

The Guide gives separate selection charts for box girders (adjacent box, spread box and tub; large segmental boxes are out of scope) and single-web girders (AASHTO I-type, also tees and double-tees). For single-web girders it recommends CFRP U-wraps to confine repairs and patches; for box girders U-wraps are generally not feasible. The charts appear later in this course. Post-tensioned members with internal tendons raise different issues, covered in BR-P03, and FRP strengthening in general is covered in BR-R01.

Assessing strength of the damaged member

Report 280 says the purpose of assessment is to reach one of three decisions: do nothing, repair the member, or replace the member. Assessment stops short of selecting the repair method. It recommends that the final assessment be made in an office away from the bridge site, with at least one experienced structural engineer taking part, preferably the same people who inspected the damage. It also asks management not to put "undue distractions or pressure" on them: "The best repair decisions are made when pressure is limited, and generally minimize overall repair time."

Two stages of calculation

During inspection, brief calculations of approximate strength are made to decide on traffic restrictions, closure or temporary supports. During assessment, a complete evaluation follows. Report 280 asks that it determine stress levels in the damaged member and compare them with both the design stresses and the allowable stresses. Service load stresses and ultimate capacity "should always be computed," and overload and fatigue stresses as appropriate. All preliminary decisions made in the field should be reviewed.

Report 280 sets two simplifying assumptions for these calculations: "assume all damaged strands as severed, and all loose or shattered concrete deleted."

A normalized rating for the damaged girder

The 2012 Guide starts from the familiar rating factor in the AASHTO Manual for Bridge Evaluation (its Eq. 6A.4.2.1-1, covered in BR-L01). Because impact damage is usually confined to one girder, it normalizes the analysis to that girder. The as-built capacity C_0 is set so that the as-built rating factor RF_0 is exactly 1. The damaged girder's rating is then:

$$RF_D = (C_D - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_P P) / (C_0 - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_P P) \text{ (Guide Eq. 2)}$$

The Guide is careful about what this number means. An RF_D below 1 "does not necessarily imply structural deficiency," as an RF below 1 from the standard equation would. It only means the girder now has less capacity than its original design. Exterior girders are often identical to more heavily loaded interior girders, to economize long-bed casting, so a damaged exterior girder may still be adequate with $RF_D < 1$. "Thus, the decision to repair, replace or do nothing to an individual girder must still be made in the context of the entire structure." The Guide itself assumes the repair objective is to restore the original capacity ($C_R \geq C_0$), while noting this may not be strictly necessary.

Residual capacity: what if the repair fails?

External repairs (bonded CFRP, external post-tensioning) can themselves be damaged by the next impact, by fire or, rarely, by vandalism. The Guide borrows the "strengthening limit" approach of ACI 440.2R: the girder without the repair must still safely carry an expected nominal load. In the absence of other guidance, for an external repair to be viable:

$$C_D \geq 1.1DC + 1.1DW + 0.75(LL+IM) \pm \gamma_P P \text{ (Guide Eq. 3)}$$

The Guide quotes ACI 440.2R's reasoning: a dead load factor of 1.1 because dead load can be assessed accurately, and a live load factor of 0.75 to exceed the statistical mean yearly maximum live load factor of 0.5. If internal strand splices are combined with external repair, the capacity with the splices alone should meet Eq. 3. A girder that fails this check is a replacement candidate if only external methods are available.

Methods of analysis

The Guide recommends plane-sections analysis, either the Whitney stress block or a fiber-section analysis satisfying strain compatibility and equilibrium, so that C_0 , C_D and C_R are computed consistently. Analyses at several sections can be "stitched together" into a capacity envelope along the span.

Effects that make damage worse than it looks

- **Adjacent strands.** With close strand spacing, there may not be enough sound concrete around strands next to a severed one to transfer their force. The Guide says it "may be prudent to disregard a portion or all of the contribution from surrounding strands" at the affected section.
- **Asymmetric box girders.** Exterior adjacent box girders often have a composite barrier and lose strands at the lower exterior corner. Both rotate the principal axes, so vertical capacity is lower than a plane-sections analysis about the horizontal axis predicts. Deteriorated shear keys can also raise the distribution factor; the Guide gives the example of an increase from 0.3 to 0.5 for adjacent box girders.
- **Prestress lost near ends or harping points.** These are critical locations. Report 280 calls the remaining force in exposed strands near an end block or harping point "a critical factor."

An effect that helps: redevelopment

Conventional practice ignores a severed strand along the whole girder. The Guide notes research showing that a severed strand "redevelops" its prestress, consistent with design transfer-length assumptions, once it re-enters sound concrete. The transfer length may be taken to start where a sound strand re-enters sound concrete. Away from the damaged zone, the strand can be counted again. This matters for long girders where damage is near the quarter point but the critical moment section is at midspan.

Measuring the force left in exposed strands

When a long length of strand has been exposed, engineers may reasonably doubt whether it has been degraded or has lost tension. Report 280 describes two field methods.

Nondestructive: the deflection method

Michigan exposed strands for about 18 ft after removing loose and shattered concrete from a severely damaged fascia girder. A small load of 20.5 lb was hung at the center of each exposed strand, and the deflection was measured from the strand above with a micrometer caliper. Treating the strand as a taut string with straight-line deflection, the tension is:

$$T = Pl / 4d$$

where P is the applied load, l the clear exposed span and d the measured deflection, in consistent units (Report 280 gives l and d in feet).

Illustrative example (not from a source). Suppose a 0.5-in. strand is exposed over 3 ft and the expected effective tension is about 25 kips (25,000 lb). With P = 20.5 lb, the expected midpoint deflection is $d = Pl/4T = (20.5)(3)/(4 \times 25,000) = 0.00062$ ft, or about 0.0074 in. That is a very small movement. The example shows why a micrometer is needed, why the reference strand must itself be stable, and why the method gives an estimate rather than a precise force. Halving the measured deflection doubles the computed tension.

Destructive: cut and measure

A length of strand can be carefully marked and cut out, and the stress calculated from its shortening. Report 280 gives the example: a 10-ft length at 144 ksi will shorten about 0.62 in., based on $E = 28,000$ ksi. (Check: $144/28,000 \times 120$ in. = 0.617 in.) It recommends an electric sander for cutting, and warns that care must be taken against whipping when strands are cut. The 2012 Guide recommends that a re-tensioning operation aim to restore about $0.60f_{pu}$, which "will generally be close to the long-term effective prestress in a strand," a useful benchmark for interpreting a measured value.

Report 280 also notes that California tried a mechanical strain gauge (a 10-in. gage length) on exposed tendons, with inconsistent results. And it adds the common-sense warning that inspection must be tied to assessment: "it would be unproductive to take strand samples if the girder will require replacement."

Other considerations in the decision

Report 280 lists further factors. Fracture critical members should get a more rigorous assessment and more conservative repairs; one way is to strengthen enough to reach zero tension in the precompressed tensile zone under all loads, and to inspect such repairs more often. Nearly all prestressed members are primary members. Diaphragms, usually reinforced concrete, are repaired like any reinforced member. A member behind a barrier or under a sidewalk may receive less overload than an interior girder, and Report 280 calls considering this "logical." It found no justification to give a repaired girder more overload capacity than its neighbors.

User inconvenience and speed of repair are treated together. Repair-in-place generally causes less inconvenience and is faster, because less is removed. If engineering assessment shows that initial traffic restrictions were too severe, they should be relaxed. Report 280 recommends establishing in advance how to obtain replacement girders, which repair-in-place techniques are acceptable, and what work state forces can do, and training repair designers "prior to having to work under emergency pressures."

Repairing concrete damage: nicks, gouges, cracks and shattered concrete

Minor nicks, spalls and scrapes

Report 280 says these "may deserve more attention than they usually get," especially in corrosive environments. Because strands are concentrated near the bottom with minimum cover, scraping away the surface finish may let corrosive elements reach the strand. It recommends "strong consideration" of cleaning the surface and sealing it with two coats of a penetrating sealer.

Gouges

The assessment depends on how the girder was designed. If it was designed for zero tension at the bottom, a minor gouge is handled like a nick: clean it and seal it with two coats of penetrating sealer. It may be packed with epoxy, but "the epoxy has no precompression and may crack at the adjacent concrete" under live load; Report 280 says it may be preferable to clean, seal and leave minor gouges open for inspection.

Girders designed to carry tension are a different case. A gouge across the bottom creates a stress raiser, and the concrete may crack below the design tension, with the crack propagating up to or above the bottom layer of strands. Report 280 recommends preloading these girders and then cleaning and inspecting the gouge. If no crack has propagated, the gouge can be packed with epoxy grout or other approved material. If a crack has propagated, concrete may have to be removed to the crack root and packed, or the crack injected and the rest packed. The preload is removed once the patch reaches strength.

Cracks

Report 280 states that cracks in prestressed members are repairable in place, and classifies them by direction (longitudinal, vertical or diagonal) and by location (inside or outside the precompressed live-load tensile zone). Its key insight concerns cracks inside that zone. Even when vertical or diagonal cracks look closed, the whole crack surface is not in compression; if it were, the cracks could not be injected. So epoxy injected there will not be precompressed and "will generally re crack when live loads are applied." The procedure: compute the live-load tensile stress (without deducting prestress); if it exceeds the allowable, as it usually will, apply preload before injecting. Longitudinal cracks, and cracks outside the precompressed tensile zone, do not need preload.

Epoxy resin can be injected into and travel along a crack only 0.002 in. (0.05 mm) wide, and Report 280 says narrower cracks are acceptable without injection. The 2012 Guide adds that only appropriately trained personnel should carry out epoxy injection.

Loose and shattered concrete

Report 280 sets no specific limit on how much loose and shattered concrete can be removed and repaired in place. It describes incidents where the entire bottom flange and web were shattered with no apparent strand damage. The only structural limitation is restoring original strength and durability. Preloading restores compression in the new concrete; external post-tensioning is another way; and a metal sleeve can be used without restoring compression, provided the stress at the top and ends of the sleeve stays within allowable tension.

Web reinforcement and displacement

Mild steel stirrups can normally be repaired in place by straightening, cutting and lap splicing, or welding. When welding, strands must be fully protected from heat and weld spatter. Report 280 notes web reinforcement is generally no larger than

No. 4 or No. 5 bars, which makes in-place repair easier.

On displacement, Report 280 says sweeping lateral curvature is "seldom caused by vehicle impacts"; it is usually locked in during manufacture, storage or construction. Abrupt lateral offsets from impact are cause for concern. Twist from impact generally cracks the girder longitudinally at the bottom of the top flange, may crack it at diaphragms, and is concentrated near the impact. Rotations are not a reason for replacement "provided the rotations are uniform in nature," and the cracks they cause are repairable. Vertical deflection is rarely changed much by impact: downward deflection may confirm many severed strands; upward deflection may indicate that strands remain in tension despite lost concrete. As a general rule, vertical deflections are too small to be of much value.

Patching practice from the 2012 Guide

- Remove all unsound concrete; remove slightly more rather than too little, unless it affects strand bond.
- Chip at least 1 in. deep, with edges as straight as possible and at right angles to the surface.
- Use the drypack method for deep holes such as core or bolt holes, mortar patches for shallow defects, and concrete replacement for voids through the section, defects beyond the reinforcement or large volumes. Concrete replacement gives the best substrate for later bonded CFRP.
- Epoxy mortars only where exceptional durability and strength are needed; latex-modified mortars help where feathered edges cannot be avoided.
- A good prepackaged patching material has an initial set of 10 to 15 minutes and a final set of 20 to 45 minutes; some compounds generate excessive heat and are not suitable for general patching.
- Preload the patch where possible, often with a vehicle parked above, to draw it into compression against shrinkage cracking and pop-out. Where preload is impractical, confine the patch with CFRP or GFRP U-wraps, topped with a UV-protective coat.

The Guide points to PCI MNL-137-06 and the ICRI/ACI Concrete Repair Manual for detailed patching guidance. General concrete repair methods are in BR-C02.

Preloading: what Report 280's tests showed

Preload is the temporary application of vertical load to the girder during repair, by jacking or, more often, a loaded vehicle. While the preload is on, the girder bends and the bottom fibers move toward tension. The patch is placed and cured in that state. When the preload is removed, the girder springs back and the new concrete is put into compression, giving it a share of precompression it would not otherwise have.

Load Test 5

Report 280's Appendix A tests were run on AASHTO Type III girders. In Test 5, four of 16 strands (25 percent) were severed and then repaired with single-strand internal splices. The procedure was to tension the splices, apply preload, inject cracks with epoxy, patch the concrete and remove the preload after the concrete gained strength. The preload required was only 26.4 kips. The repaired girder's cracking load was 49.7 kips, against 55.1 kips for the original uncracked girder. Report 280 says the difference "could easily be explained by not having injected the cracks with epoxy resin" and concludes that Test 5 "proved the validity of restoring compression through the use of preload."

Report 280 says the basis for the amount of preload should be a stress calculation, and that generally enough compression can be restored to equal the $LL + I$ tension. It cautions against more: the test girder was loaded to about twice $LL + I$, cracks

reached within a few inches of the slab and then closed, and preloading enough to prevent cracking at that overload "should not be necessary and is not desirable."

The 2012 view

The Guide is more cautious about preload as a way to restore prestress force. Preload is most effective for smaller girders; as girders get larger the preload needed "becomes very large and often impractical," and the effectiveness improves only with lower dead-to-live load ratios, which are not typical of concrete bridges. Its conclusion: preloading to restore prestress force, "while feasible, is typically impractical," but preloading to precompress patch repairs is recommended. It also warns not to overload the structure or create damaging local stresses with the preload.

Illustrative note (not from a source). A useful way to see the size problem: preload works by imposing a moment on the girder. To create a given compressive stress at the soffit in a deep bulb-tee, the moment required scales with its section modulus, which can be several times that of a Type III girder. The vehicle that was enough for the Type III test may not come close on a long-span girder, and stacking enough load on one girder may overload its neighbors or the deck. That is the practical reason the Guide prefers active methods for restoring prestress in large girders.

Restoring strands: splices, sleeves and external post-tensioning

When strands are severed, Report 280 found that "nearly all state bridge engineers have some minimum number of severed strands that they judge as being permissible, without splicing," and that this judgment "should be based on a stress calculation." All or nearly all states then allowed one severed strand, and few if any allowed more than three. Report 280 saw no need for an arbitrary upper limit on the number that can be spliced; the limit is set by the capacity of each splicing technique.

Internal strand splices

Internal splices reconnect a severed strand inside the girder. The 2012 Guide describes commercially available splices with a coupler joining reverse-threaded anchors: turning the coupler draws the anchors together and tensions the strand, much like a turnbuckle. Report 280 found these splices "inexpensive and easy to install," and noted they can be torqued to about the tension in adjacent unsevered strands. Preload must still be used in the spliced area to restore compression in the patch.

Limits from the two sources:

- **Fatigue.** Report 280 cites University of Texas studies showing the couplers reduce fatigue life, and advises that ordinary splices be used conservatively. Limiting splicing to 25 percent of the strands "may be a conservative figure." If all splices failed, the stress increase would then be 33 percent (one remaining strand carrying the share of $1/0.75 = 1.33$ strands).
- **Strength.** The Guide notes commercial literature implies $0.96f_{pu}$, while testing has reported a "guaranteed" strength of $0.85f_{pu}$, with other research reporting anywhere from about 80 to 100 percent. It recommends limiting splices to developing $0.85f_{pu}$ and taking the effective number of restored strands as **$0.85 n_{spliced}$** in ultimate calculations.
- **Size and geometry.** Splices are available only for strand up to 0.5 in. diameter. Anchor bodies are 1.625 in. in diameter regardless of strand size, while strands are often at 1.5 in. centers. Adjacent splices must therefore be staggered, with at least 2 in. recommended to allow for the coupler's stroke. Splicing exterior strands reduces cover at the splice unless the section is enlarged.

- **Box girders.** Beyond a few isolated strands, the Guide does not recommend splicing in box girders, especially older ones, because of thin cover, tight spacing and the inaccessible top of the bottom flange inside the void. Splicing is more practical in flanged (I-shaped) members.
- **Re-tensioning target.** The Guide suggests aiming for about $0.60f_{pu}$. Strand heating to tension a splice is not recommended.

Metal sleeve splice

Report 280 developed a galvanized A36 steel sleeve, bonded to the girder with injected epoxy, that can splice a large number of severed strands and also suits girders with large volumes of loose or shattered concrete. Its detailing rules:

- Extend the sleeve at least 3 ft 3 in. beyond the damaged concrete.
- For six strands or fewer, lap the severed strands at least 5 ft 3 in.
- For more than six strands, lap at least 160 strand diameters.
- Preload is not needed provided the stress at the top and ends of the sleeve is within the allowable for all loads. Bond stress could govern how many strands can be spliced.

The 2012 Guide later judged steel jacket repairs "very cumbersome and impractical" and focused instead on CFRP, which addresses most of their drawbacks.

External post-tensioning with steel

External steel post-tensioning (PT-steel) uses high-strength bars or strand anchored at corbels or bolsters cast or mounted on the girder, usually on the web. Report 280 used 150-ksi threaded bars (ASTM A722) or seven-wire strand, and found that corbels strong enough to splice four 0.5-in. 270-ksi strands on an AASHTO Type III girder "can be readily constructed," but that splicing more than six to eight strands this way "might be difficult." Corbel strength generally controls.

The Guide adds that any degree of damage may be repaired with PT-steel, and the primary design problem is getting the force into the girder through the bolsters, designed as brackets or corbels, often themselves post-tensioned through the web. Bolts must not hit sound strands. PT-steel can also run between girders anchored to diaphragms, with king- or queen-posts to harp it. But PT-steel has a higher profile and "may be susceptible to brittle and catastrophic failure in the event of a subsequent impact," so it is inappropriate on the soffit.

FRP repair methods and how to choose

The 2012 Guide's main addition to Report 280 is fiber-reinforced polymer repair. It evaluated nine techniques and generally recommends four: externally bonded CFRP (EB-CFRP), bonded post-tensioned CFRP (bPT-CFRP), PT-steel and internal strand splicing, plus hybrids of external repair with strand splices.

EB-CFRP

Preformed CFRP strips bonded to the soffit or web. The Guide recommends EB-CFRP for girders that do not need prestress restored (SEVERE I and II). Preformed strips are preferred over wet lay-up fabric for flexural repair. Its representative high-strength CFRP strip properties are a modulus of 23,200 ksi, strength of 406 ksi and rupture strain of 0.017, typically 0.047 in. thick in 2-, 3- and 4-in. widths. The governing limit state is debonding, which in a sound application is a cohesive failure through a thin layer of cover concrete, so a better adhesive does not raise the limit much. Debonding starts at the material

closest to the soffit, so material on the upper web is less efficient. The Guide proposes environmental reduction factors of 0.90 on CFRP strip properties and bond for exterior bridges, versus ACI 440.2R's 0.85 for CFRP.

PT-CFRP

Stressing the CFRP puts some lost prestress back and raises the effective debonding strain. The Guide recommends **bonded** PT-CFRP for damage needing restored prestress (SEVERE III). It does not recommend prestressed (pre-bonded) CFRP, because of complex application for marginal benefit, or unbonded PT-CFRP, because of fretting under traffic. It suggests keeping the effective CFRP prestress level κ below 0.50 for bonded systems. Anchorages are proprietary; shear-key anchorages should carry 100 percent of the prestress force.

Geometry limits

- Soffit repairs are most efficient but must not cut vertical clearance; the Guide considers a 1-in. envelope reasonable.
- Only EB-CFRP or PT-CFRP suit soffits in most cases; fully bonded CFRP also makes the girder more resilient to a later impact.
- For adjacent box girders only the soffit is available.
- U-wraps are recommended on single-web (I) girders to confine patches and hold longitudinal CFRP, but are generally not feasible on boxes.

Traffic during repair

The Guide recommends keeping traffic off the girder being repaired during installation, and for bonded systems restricting traffic during the initial epoxy cure (typically about three hours) and for 24 hours if possible. Because damage usually affects exterior girders, closing the outside lane is generally enough; closing it to trucks while allowing small vehicles is likely sufficient, and the adhesive manufacturer's guidance should be followed.

Hybrid repairs

Strand splices are internal, so they can be combined with most external methods (possibly not NSM, where splice chucks and slots interfere). Counting $0.85 n_{\text{spliced}}$, the splices reduce the damage the external repair must cover, and because they are active, they can restore enough prestress to avoid decompression and allow a passive external repair. The Guide notes that other hybrid combinations are generally inefficient because their stiffnesses differ.

The selection flow charts

Class	Single-web (I) girders: recommended	Box girders: recommended
MINOR, MODERATE	Patching and corrosion mitigation; no structural repair	Same
SEVERE I	EB-CFRP strip, strand splice	EB-CFRP strip, strand splice
SEVERE II	EB-CFRP strip, PT-steel, strand splice	EB-CFRP strip
SEVERE III	PT-steel, bPT-CFRP, hybrid with strand splice	bPT-CFRP
SEVERE IV	Replace girder	Replace girder

This is a simplified reading of the Guide's Figures 2 and 3, which also list "viable" alternatives such as CFRP fabric, NSM-CFRP and, for boxes, PT-steel or hybrids at SEVERE III. Consult the figures directly before using them.

Corrosion, replacement and a worked example

Corrosion mitigation

The Guide says the source of corrosion must be dealt with before patching and repair. Chlorides can come from deicing salt through poor drainage or failed joints, from traffic spray below, or from a marine environment. Adjacent box girders with non-composite decks can leak through longitudinal joints and "wick" along the soffit. Surface treatments are effective in the short term but effective long term only if applied before chloride contamination. FRP wraps also help by excluding chloride-bearing water. Passive cathodic protection (embedded anodes in patches) is effective. Impressed-current systems work too but are rarely used because of cost and complexity, and if polarization is excessive, atomic hydrogen can form at the steel and cause hydrogen embrittlement of the strand.

Replacement

Report 280 calls complete replacement "normally the most expensive method of repair." It requires removing part of the slab, and is harder still for girders made continuous for live load. It notes Minnesota DOT used a method of replacing simple-span girders with minimal slab and diaphragm removal, and Report 226 records replacement from below and one replacement with a precast girder-slab-curb section. Report 280 recommends establishing in advance how replacement girders will be procured. Report 226 notes that some agencies had ongoing agreements with manufacturers for emergency replacement girders.

After repair

Report 280 asks that calculations show the repaired girder has essentially the same service and ultimate capacity as the original, with minor differences accepted if justified. Repairs on fracture critical members should be inspected more frequently, and any repair should be added to the bridge file so later inspectors know what to look for.

Illustrative worked example (not from a source)

All numbers below are invented for teaching. An overheight truck strikes the fascia I-girder of a simple span carrying a four-lane road over an interstate, about 30 ft from one bearing on a 100-ft span. The girder has 40 bottom strands of 0.5-in. diameter; none are harped in the damaged zone. The inspection report records a 6-ft length of shattered bottom flange, 4 strands severed, 2 flattened and 3 exposed but intact, closed hairline vertical cracks nearby, no measurable lateral offset and no visible sag.

1. **Strand count for assessment.** Treat the 2 flattened strands as severed (Report 280): 6 of 40, or 15 percent. Consider whether strands next to the severed ones can still transfer force; with closely spaced strands in shattered concrete, conservatively discount the 3 exposed strands at this section as well.
2. **Classification.** 15 percent is above 5 and below about 20, with no sag: provisionally SEVERE II under Guide Table 1. That points to a strength repair; prestress restoration may not be needed if a decompression check confirms the soffit stays in compression under service load.
3. **Location.** The damage is near the third point, not at midspan. Redevelopment means the severed strands can be counted again a transfer length past where they re-enter sound concrete. Build a capacity envelope rather than checking midspan only.
4. **Normalized rating.** Compute C_D and RF_D . If the exterior girder was cast identical to more heavily loaded interior girders, it may still rate adequately even with $RF_D < 1$.
5. **Residual capacity.** Check $C_D \geq 1.1DC + 1.1DW + 0.75(LL+IM)$. If it passes, an external repair is viable.

6. **Selection.** For a SEVERE II I-girder, candidates are EB-CFRP strips, PT-steel or strand splices. Splicing 6 strands at 0.85 credit restores about 5.1 strand-equivalents and is 15 percent of the strands, within Report 280's conservative 25 percent figure. A splice-plus-patch with U-wrap confinement, or EB-CFRP on a concrete-replacement patch, would both be reasonable to evaluate. PT-steel is less attractive on the soffit over interstate traffic because of clearance and its vulnerability to another hit.
7. **Durability.** Remove unsound concrete at least 1 in. deep, preload or confine the patch, check chloride sources, consider embedded anodes, and coat exposed FRP.
8. **Records.** Document the decision and calculations, and set a follow-up inspection interval.

Checklist for practice

- Is a damage-response procedure, including emergency girder procurement, in place before the next strike?
- Did the inspection record every strand condition, crack widths and directions, and displacements, without recommending a repair?
- Were damaged strands treated as severed and loose concrete as removed?
- Were adjacent-strand effects, asymmetry (for boxes), harping points and girder ends considered?
- Does the chosen repair restore the right limit state: strength only, or service too?
- Does the girder pass the residual capacity check if the external repair is lost?
- Has every exposed strand been covered by a sound, preferably precompressed or confined, patch?

Key takeaways

- Impact damage concentrates on the bottom strands at minimum cover, and replaced concrete carries no precompression unless preload or active post-tensioning restores it.
- Separate inspection from assessment: the inspection report records facts and should not normally recommend a repair.
- For strength calculations, treat all damaged strands as severed and all loose or shattered concrete as removed (Report 280).
- The 2012 Guide classifies damage from MINOR to SEVERE IV; roughly, more than 5% strand loss needs structural repair, decompression (about 20% or more) needs restored prestress, and more than 35% or vertical deflection over 0.5% means replacement.
- External repairs are viable only if the damaged girder alone meets the residual capacity limit $C_D \geq 1.1DC + 1.1DW + 0.75(LL+IM)$.
- Recommended repairs are EB-CFRP, bonded PT-CFRP, PT-steel and internal strand splices (credited at 0.85 per spliced strand), with every exposed strand patched.

References

1. NCHRP Report 280, Guidelines for Evaluation and Repair of Prestressed Concrete Bridge Members, 1985
2. NCHRP Report 226, Damage Evaluation and Repair Methods for Prestressed Concrete Bridge Members, 1980
3. PCI, Bridge Design Manual, 3rd Edition (with revisions)
4. NCHRP Project 20-07, Task 307: Harries, K.A., Kasan, J., Miller, R., Brinkman, R., Guide to Recommended Practice for the Repair of Impact-Damaged Prestressed Concrete Bridge Girders, Final Report Appendix A, University of Pittsburgh / AASHTO, 2012.
5. ACI 440.2R, Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures (as cited by the 2012 Guide; 2008 edition).

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