GUIDE TO RECOMMENDED PRACTICE FOR THE REPAIR OF IMPACT-DAMAGED PRESTRESSED CONCRETE BRIDGE GIRDERS

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1 INTRODUCTION

Throughout this *Guide*, commentary and reference to the supporting NCHRP 20-07 Task 307 Final Report (referred to as the *Report*) are made by section number in square brackets.

This *Guide* serves to update the 1985 *NCHRP Report 280: Guidelines for Evaluation and Repair of Prestressed Concrete Bridge Members*. This report remains a primary reference for this topic. Material repeated from *NCHRP 280* is duly cited.

1.1 Significance and Scope

This document provides guidance for inspecting, assessing and repairing damage to prestressed concrete bridge girders resulting from vehicular impact. This *Guide* focuses on structural (load carrying) repair techniques rather than aesthetic or preventative repairs. Guidance for the latter is given by reference to other established sources. Similarly, the focus of this *Guide* is impact damage, although the repair methods described may also be employed for similar damage from other sources.

This *Guide* is written considering the following underlying assumptions:

Repair methods are intended to restore all or a portion of ***individual girder*** capacity lost due to impact damage and subsequent impact-related deterioration. This *Guide* does not consider strengthening beyond the original undamaged girder capacity although the methods may be appropriate for this objective in some cases.

Repair methods are intended to be 'permanent'. With proper details and maintenance the repair method becomes a permanent part of the girder and is expected to have a life equal to that of the girder. Temporary repair techniques are beyond the scope of this *Guide*.

Combinations of impact damage, bridge geometry, extant condition and material properties are necessarily unique and must be considered on a case-by-case basis. This *Guide* attempts to delineate an approach to repair of such damage emphasizing the applicability, utility and limitation of repair techniques.

1.2 Factors Affecting Impact Damage

Impact damage is usually readily apparent and varies from minor scrapes to structural collapse. The most commonly occurring damage pattern associated with side impact to prestressed concrete girders causes a torsion-induced shear cracking pattern in the exterior (or fascia) girder as shown in Figure 1a. In cases where the impact is more direct, this pattern becomes more of a shear push-out (Figure 1d). Examples of impact damage are provided in Section 1.3.2 through 1.3.4 of the *Report*.

a) typical impact damage due to side impact (Feldman et al. 1996)

b) typical impact damage pattern (PennDOT)

c) typical impact damage pattern

d) typical shear push-out damage pattern

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Figure 1 Examples of typical damage due to vehicle impact.

The following address additional factors affecting the nature of damage in the context of repair.

1.2.1 Corrosion Damage Subsequent to Impact

Left uncorrected, minor damage (nicks and scrapes) may progress to becoming more significant as corrosion becomes manifest. Eventually corrosion can lead to section loss of the strand and resulting loss of prestress and member capacity [1.3.1.1]. In general, the progression of corrosion-related damage tends to be exponential in time. Repairing such damage must be accompanied by mitigating the damage where possible.

The source of corrosion-inducing chlorides may vary. Chlorides from de-icing salts or salt-water evironments may be introduced from the bridge deck as a result of poor drainage, damaged joints or other sources of damage. Adjacent box girder structures having a non-composite deck, for instance, may experience deck drainage through longitudinal joints and 'wicking' along the girder soffit affecting a large region of potentially susceptible concrete. Chlorides may also be introduced as a result of spray from traffic passing beneath the bridge or from a salt water environment, affecting the entire soffit region. Box girders may also be susceptible to chloride ingress from the top of the bottom flange if water is able to penetrate the girder cells, although this is believed to be rare.

1.2.2 Adjacent Strands

Impact damage to prestressed concrete girders may be more severe than visually apparent. Small strand spacing may result in insufficient concrete surrounding strands adjacent to damaged or severed strand(s) to allow the prestressing force of these undamaged strands to be transferred into the structure. As a result, a portion or all of the prestressing force near the impact may be ineffective. It may be prudent to disregard a portion or all of the contribution from surrounding strands at the affected section in a repair design [1.3.1.2].

1.2.3 Strand 'Redevelopment'

Damaged strands in larger spans or long girders may be 'redeveloped' if there is sufficient undamaged length remaining. Conventional practice conservatively neglects any severed strand along the entire length of the girder in the analysis of the structure. However it has been shown that a severed strand 'redevelops' prestressing force in a manner consistent with the transfer length assumptions used in design (AASHTO 2010) once the damaged strand re-enters sound concrete [1.3.1.3 and 2.5].

1.2.4 Unanticipated Composite Action

Exterior adjacent box girders often have a composite barrier wall resulting in an asymmetric section and load condition; and have asymmetric strand loss concentrated on their lower exterior corner. These effects individually (and more so in combination) result in a rotation of the principal axes of the section [1.3.1.5]. As a result, the capacity of the girder to resist moment applied about its geometric horizontal axis will be reduced beyond what is predicted using a typical plane sections analysis due to the biaxial nature of the bending. In adjacent box bridges, this effect is most pronounced in exterior girders since interior girders are restrained from rotation by adjacent girders. This behavior becomes significant when performing load rating of the structure. Occasionally, the girder and barrier wall are assumed to be composite and a sections analysis is performed about the horizontal axis. Doing so will overestimate the true vertical capacity since it does not account for the biaxial response of the section [1.3.1.4].

Additionally, many adjacent box bridges have deteriorated or non-existent shear keys, resulting in the girders behaving independently and a corresponding increase in the transverse live load distribution factor (from 0.3 to 0.5 for adjacent box girders, for instance). Kasan and Harries (2012) present an analytical approach shown to capture anticipated eccentric behavior where it exists [2.1.1].

1.2.5 Lateral Deflections and Twist of Girders

Lateral sweep of a prestressed girder is seldom caused by impact-damage but rather is a result of fabrication, storage or construction errors. Lateral sweep due to vehicle impact is typically abrupt and localized to the area near the impact. Permanent twist or rotation can be caused by vehicle impacts or faulty manufacturing, storage or construction. Twist due to impact loading will generally crack the girder longitudinally at the bottom of the top flange (Figure 1) and at diaphragm locations. Permanent twist due to impact loading will be concentrated in the area of impact [1.3.1.6].

1.2.6 Effect of Diaphragms on Impact Damage

Many prestressed structures utilize intermediate diaphragms between beams for lateral stability. When a girder is struck by an over-height vehicle, the diaphragms transfer the impact load to the adjacent members. It has been shown that, in general, structures with intermediate diaphragm perform favorably when subject to impact, by spreading the impact load to adjacent girders. Nonetheless, shallow diaphragms may result in increased damage as the impacted girder flange rotates about the diaphragm; therefore full depth intermediate diaphragms are recommended. For a long bridge, multiple and distributed intermediate diaphragms resist impact better by effectively transferring large deformations to other girders and the deck, reducing the damaged areas and absorbing more kinetic energy [1.3.1.7].

2 INSPECTION OF IMPACT DAMAGED PRESTRESSED GIRDERS

All inspection should be guided by the *AASHTO Guide Manual for Bridge Element Inspection* (2011). In general, visual and manual inspection is the only practical triage tool for impact-affected prestressed concrete girders [1.4]. A significant goal of this triage is to identify locations requiring further non-destructive or destructive evaluation. A skilled inspector, familiar with the structure is able to provide a remarkably accurate assessment of the condition of the structure although is unlikely to be able to accurately quantify many damage types.

The goal of the visual inspection is to identify damages to the girder which affect the girder's load carrying ability. Visual indicators include:

- exposed/corroded/severed strands
- longitudinal and transverse cracks
- concrete spalling
- efflorescence
- rust staining
- presence of water/leakage
- longitudinal cracks on deck
- evidence of displacement between beams
- web out-of plumbness
- relative dislocation of girder from bearing
- shear or flexure cracking

When documenting damage in the inspection report, every effort should be made to obtain photographic evidence of the damage in order to provide a visual for the assessment engineer and to provide a basis for inspection-to-inspection comparison. When documenting cracks, crack orientation and severity (width) are vital and must be documented. Other pertinent items should be noted as needed, such as a source of damage exacerbation (i.e.: improperly functioning drainage systems over the damage location). Ahlborn (2005) provides an inspection handbook for adjacent box girder structures which illustrates typical forms of distress for this bridge type.

Surface tapping and using a chipping hammer to remove loose concrete are common and recommended methods of manual inspection. With great care, the 'screw-driver test' (Walther and Hillemeier 2008) may be used to inspect the interior wires of prestressing strand and to assess the degree of remaining prestress [1.4.1].

Where corrosion of non-visible reinforcement or strand is suspected, the surface potential survey/half-cell potential survey [1.4.2] is a well-established standardized inspection technique (ASTM C876). While cumbersome, it is presently the most viable and widely used *in situ* approach alongside visual and other manual forms of inspection.

Remnant magnetism [1.4.3] is a useful method to get information about the location of prestressing steel fractures and the degree of damage to a strand. Presently, commercially available systems are aimed at detection of flaws/damage in prestressed slabs although could be readily adapted to high-speed applications on bridge soffits. This technique is very promising in near-term.

The acoustic emission (AE) technique is a noninvasive, nondestructive method that analyzes noises ("events") that are created when materials (i.e. concrete or prestressing steel) deform or fracture [1.4.4]. The method is applicable for real-time health monitoring of a bridge or a girder and has been successfully used to quantify and precisely locate damage in prestressed concrete girders. Alternate approaches using known applied loads (trucks) have been demonstrated to be viable methods of inspection for structures having significant existing damage, although some technical hurdles remain before wide-spread deployment is practical. AE methods are presently 'baseline' techniques, that is, they are unable to capture damage occurring before monitoring is initiated.

While there are numerous other inspection techniques available to the practitioner [1.4], few are practically deployable or offer great utility for the inspection of impact damaged prestressed concrete girders.

It is stressed that inspection is not member assessment. The goal of inspection is to communicate information pertaining to the structure's condition and damage severity and location which could affect the member's load carrying capacity to the party responsible for assessment. Additionally, the importance of the inspection should not be overlooked as the capacity assessment relies on the information gathered during inspection.

3 ASSESSING (RATING) IMPACT-DAMAGED PRESTRESSED GIRDERS

3.1 Rating Impact Damaged and Subsequently Repaired Girders

AASHTO *Evaluation Manual* (2011) Eq. 6A.4.2.1-1 provides the basis for the bridge rating factor, RF:

$$\mathrm{RF} = \frac{\mathrm{C} - \gamma_{\mathrm{DC}}\mathrm{DC} - \gamma_{\mathrm{DW}}\mathrm{DW} \pm \gamma_{\mathrm{P}}\mathrm{P}}{\gamma_{\mathrm{LL}}(\mathrm{LL} + \mathrm{IM})} \qquad \text{(Eq. 1)}$$

Where C is the structural capacity, DC, DW, LL, IM and P are load effects prescribed in the AASHTO *LRFD Bridge Design Specifications* (2010), and the values of γ are LRFD load factors prescribed in Table 6A.4.2.2 of the *Evaluation Manual* (2011). These factors differ for inventory and operational rating levels.

Because impact damage is very often localized to one girder – usually an exterior girder – adjusting this approach to consider only the damaged girder is convenient and appropriate [2.1.2]. In this approach, the capacity of the as-built girder corresponds exactly to $RF_0 = 1$; that is: $C_0 = \gamma_{DC}DC + \gamma_{DW}DW + \gamma_{LL}(LL+IM) \pm \gamma_P P$, and the existing or damaged capacity is C_D, then the ***normalized rating factor*** for the damaged girder is:

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

In this manner analyses are effectively normalized by the AASHTO-prescribed inventory RF value (Eq. 1) and the normalized undamaged girder rating factor $RF_0 = 1.0$. If RF for the as-built structure is known (RF_0), then the C_0 term in Equation 2 may be replaced with RF_0C_0. In either case, ratings may proceed since the objective of the rating is to consider the capacity of the repaired girder (C_R) relative to C_D and a target capacity C_0 (or another specified capacity). A rating factor less than unity based on Equation 2 does not necessarily imply structural deficiency as is the case when a rating factor less than unity is found from Equation 1. A value of less than unity from Eq. 2 simply indicates that the girder capacity is lower than its original design capacity. If the girder has excess capacity, it may still be adequate with a relative $RF_D < 1$. This situation is relatively common where identical girders are used for interior and more lightly loaded exterior girders in order to economize long-bed prestressing operations. Thus, the decision to repair, replace or do nothing to an individual girder must still be made in the context of the entire structure.

3.1.1 Repair Objective

This *Guide* is prepared on the assumption that the goal of a repair is to restore the original undamaged capacity of an impact-damaged girder (i.e.: $C_R \geq C_0$). Clearly, this may not be strictly necessary to the overall performance of the bridge. In this sense, Equation 2 is most useful in assessing the relative effect of the repair provided with respect to the individual girder capacity. These values are then considered in the overall bridge rating. The bridge rating process is necessarily unique to each bridge and beyond the scope of this *Guide*.

3.1.2 Residual Capacity and Strengthening Limits

External repair techniques are subject to damage from subsequent impact, fire or, in rare cases, vandalism. Therefore limits to the strengthening effect of a repair should be considered (ACI 440-2R 2008). The residual capacity of the unstrengthened girder (i.e.: the damaged capacity C_D) should safely resist an expected nominal load. In the absence of alternate guidance, for an external repair to be viable [2.3.1]:

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

If internal strand splicing is combined with external repairs (so called 'hybrid' repairs; see Section 6.5), the capacity of the girder considering the strand splices alone should exceed the limit given by Eq. 3.

3.2 Methods of Analysis

Due to the nature of prestressed member repairs and the need to assess the undamaged (C_0), damaged (C_D) and repaired (C_R) capacities in a consistent manner, plane sections analyses are most appropriate. Traditional Whitney stress-block approaches or fiber sections analyses satisfying strain compatibility and equilibrium are recommended. These analyses are conducted at various sections along a girder and may be 'stitched together' to create a capacity envelope [2.5]. Transitioning between damaged and undamaged sections may be considered using AASHTO-prescribed transfer and development lengths in the same manner as one treats the ends of girders. The transfer length may be assumed to begin at a location that a sound strand reenters sound concrete [1.3.1.3 and 2.5].

3.2.1 Non-composite Exterior Box Girders with Asymmetric Damage and Loading

Non-composite exterior box girders are generally loaded in an asymmetric manner and have some degree of asymmetric load resistance associated with typical impact patterns [1.3.1.4]. Additionally, deteriorated shear keys may result in increased live load distribution factors from those assumed in design [2.1.1]. Asymmetry results in a rotation of the neutral axis and related reduction in vertical load carrying capacity resulting from the unanticipated coupled transverse flexure. Kasan and Harries (2012) present an analytical approach shown to capture such eccentric behavior where it exists [2.1.1].

Such effects have been shown to be negligible for sections other than box girders and for interior girders that are restrained from rotation by adjacent girders.

3.3 Damage Classification of Prestressed Concrete Bridge Girders

Damage to impact-damaged prestressed concrete girders is described in a spectra ranging from minor to severe as broadly described below. Damage classification must be considered on a case-by-case basis. The classifications given in Table 1 are intended to provide approximate guidance only for the classification of damage to prestressed concrete girders.

Table 1 Damage classification for prestressed concrete girders

		strand loss	**camber**
MINOR	Concrete with shallow spalls, nicks and cracks, scrapes and some efflorescence, rust or water stains. Damage does not affect member capacity. Repairs are for aesthetic and preventative purposes only (*NCHRP 280*).	no exposed strands	no effect of girder camber
MODERATE	Larger cracks and sufficient spalling or loss of concrete to expose strands. Damage does not affect member capacity. Repairs are intended to prevent further deterioration (*NCHRP 280*).	exposed strands no severed strands	no effect of girder camber
SEVERE I	Damage affects member capacity but may not be critical – being sufficiently minor or not located at a critical section along the span [2.5]. Repairs to prevent further deterioration are warranted although structural repair is typically not required.	less than 5% strand loss	partial loss of camber
SEVERE II	Damage requires structural repair that can be affected using a non-prestressed/post-tensioned method. This may be considered as repair to affect the STRENGTH (or ultimate) limit state.	strand loss greater than 5%	complete loss of camber
SEVERE III	Decompression of the tensile soffit has resulted [2.6.1.2]. Damage requires structural repair involving replacement of prestressing force through new prestress or post-tensioning. This may be considered as repair to affect the SERVICE limit state in addition to the STRENGTH limit state.	strand loss exceeding 20%. In longer and heavily loaded sections, decompression may not occur until close to 30% strand loss.	vertical deflection less than 0.5%
SEVERE IV	Damage is too extensive. Repair is not practical and the element must be replaced.	strand loss greater than 35%	vertical deflection greater than 0.5%

3.3.1 'Repair or Replace' Threshold

The threshold between SEVERE III and SEVERE IV essentially represents the 'repair or replace' criteria. In addition to the general guidance provided in Table 1, the following conditions require girder replacement [1.5.2]:

- Permanent lateral deflection exceeding standard girder tolerance (*NCHRP 280*) [1.3.1.6]
- Permanent vertical deflection from horizontal exceeding 0.5%
- Cracks at the web/flange interface that remain open indicating yield of transverse steel
- Loss of prestress at harping point [3.2 (6)]
- Loss of prestress at girder ends exceeding 25% of the total number of strands (*AASHTO LRFD 5.11.4.3*) [2.5.1]
- Damage girder capacity falling below the residual capacity given by Eq. 3 if external repair methods are used. If strand splicing is combined with external repairs, the capacity of the girder repaired with strand splices alone should exceed the limit given by Eq. 3.

3.3.2 'Repair or Do Nothing' Threshold

From the perspective of structural (load bearing) repair, the threshold between SEVERE I and SEVERE II has few clear delineators and will generally by based on the judgment of the design professional. Often the loss of a few discrete strands in a section will not warrant structural repair. CFRP repair techniques have an effective lower limit of their utility. It is not practical to use a CFRP technique when damage does not exceed this limit as described in Section 6.2.4.1. [2.3.3.1]

Wherever possible, strand, whether severed or in good shape should not remain exposed to the environment as deterioration will accelerate and propagate. Exposed strand conditions should be mitigated using sound patching techniques as described in Section 7.

4 REPAIR OF PRESTRESSED CONCRETE BOX GIRDERS

Box girders include adjacent box, spread box and tub girders. Large segmental-type box girders are not within the scope of this *Guide*.

Damage classification must be considered on a case-by-case basis. Table 1 is intended to provide approximate guidance only for the classification of damage to adjacent and spread prestressed concrete box girders. MINOR and MODERATE damage does not require structural (load bearing) repair and is addressed in Section 3.3 and 7.1.

Selection of recommended repair techniques for prestressed concrete box girders is shown schematically in Figure 2. Repair techniques, their application and limitations are described in Section 6. It is generally not feasible to provide additional confinement (i.e.: CFRP U-wraps) to box girder repairs.

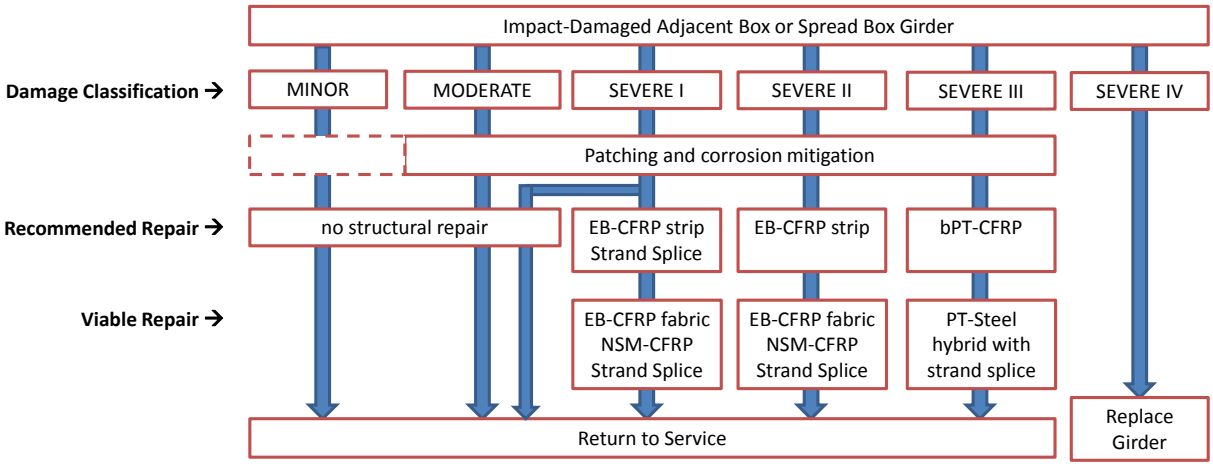


Figure 2 Repair selection flow chart for prestressed concrete box girders.

5 REPAIR OF PRESTRESSED CONCRETE SINGLE-WEB GIRDERS

Single web girders include flanged girders similar in form to AASHTO-I sections. Guidance is generally applicable to non-flanged sections such as tees and double-tees.

Damage classification must be considered on a case-by-case basis. Table 1 is intended to provide approximate guidance only for the classification of damage to single-web prestressed concrete girders. MINOR and MODERATE damage does not require structural (load bearing) repair and is addressed in Section 3.3 and 7.1.

Selection of recommended repair techniques for prestressed concrete single-web girders is shown schematically in Figure 3. Repair techniques, their application and limitations are described in Section 6. It is recommended to provide additional confinement (i.e.: CFRP U-wraps) to single-web girder repairs; this also provides confinement to any necessary concrete patches.

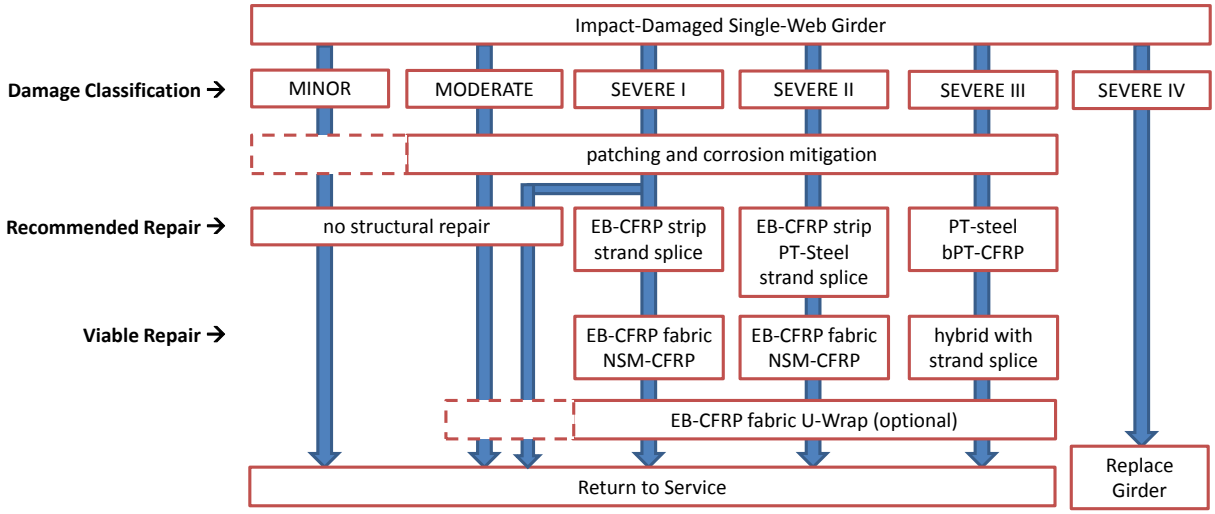


Figure 3 Repair selection flow chart for prestressed concrete single-web girders.

6 PRESTRESSED GIRDER REPAIR TECHNIQUES

The following sections describe techniques for the structural (load bearing) repair of impact-damaged prestressed concrete girders. Non-structural repair applications are described in Section 7.

6.1 Limitations of Repair Techniques Associated with Damaged Girder Geometry

Bonded repair techniques may be applied to accessible regions of the girder soffit and web(s). Soffit repairs are more efficient since these necessarily place the repair material as far from the member neutral axis as possible. However, soffit repairs must not encroach on the vertical clearance below the bridge. It is felt that an envelope of 1 inch for such repairs is reasonable unless other remedial action is taken such as lowering the roadway elevation beneath the bridge. Only externally bonded CFRP (EB-CFRP; see Section 6.2.4) or post-tensioned CFRP (PT-CFRP; see Section 6.2.5) repairs are suitable for soffit applications in most cases. Fully bonded CFRP applications (EB-CFRP or bPT-CFRP) also improve the resilience of the member in the event of a subsequent vehicle impact [1.5.4.6]. Post tensioned steel (PT-steel; see Section 6.3.1) applications will generally have a higher profile and may be susceptible to brittle and catastrophic failure in the event of a subsequent impact making them inappropriate for soffit applications. For adjacent box (AB) structures, only the soffit is available for external repair.

For bridge systems where the girder webs are accessible, bonded repair to the webs is feasible and does help to address the potential damage associated with subsequent impacts. However, as the repair material is located closer to the neutral axis of the member, its structural efficiency and therefore contribution to the load carrying capacity of the girder is diminished, requiring more material to be used to accomplish the same strengthening objective. Furthermore, for bonded repairs, the failure limit state is typically debonding of the CFRP which must be understood to occur *at the extreme tension element* of the member. As a result, failure occurs when material bonded at, or closest to the soffit debonds. Once debonding initiates in this fashion, load is redistributed to the internal prestressing strand and the CFRP located away from the soffit – driving a progressive debonding failure of this latter material. This behavior results in further diminished efficiency of material bonded to the web away from the soffit since the CFRP strain is limited *at the level of the soffit* [2.3.2].

External post-tensioned systems are more easily used in bridge systems where the webs are accessible. PT-steel systems in particular, may be attached to the girder webs using bolsters (Figure 4a) or be located between girders, anchored to new or existing diaphragms (Figure 4b). In these cases, installation of king- or queen-post to harp the PT strand is feasible and may be used to improve the efficiency of the repair (the example shown in Figure 4b shows a small degree of harping between the anchorage and subsequent diaphragm supports).

a) PT-steel anchored to girder using steel bolster bolted through girder web (second PT bolster on back of web) [DYWIDAG Systems International]

b) PT-CFRP anchored and harped by diaphragms [Mamlouk and Zaniewski *Materials for Civil and Construction Engineers* 2011]

Figure 4 Anchorage of post-tensioned repair systems.

6.1.1 Repair Applications During Live Loading

In practice, there are many situations under which deteriorated structures are subjected to continuous transient vehicles loads. Since repair applications are generally located on the underside of the bridge, it is particularly beneficial if the repair can be implemented without interrupting the traffic on the bridge.

Research on the performance of repair techniques installed under conditions of continuous vehicle loading acting during the installation are rare and their results contradictory [1.5.3.6]. It is recommended that traffic be restricted from affecting the girder being repaired during the repair installation. For externally bonded systems relying on an adhesive interface (EB-CFRP and bPT-CFRP), it is recommended that traffic be further restricted during the initial epoxy cure (typically about three hours) and for 24 hours if possible. This is not to say that a bridge must be closed during repair. Impact damage typically affects exterior girders. Closing the outside lane to traffic should be sufficient to effectively install most repairs. Limited research suggests that typical levels of transient strain does not affect epoxy cure and that only large strain excursions should be avoided. In this case, closing the affected lane to

trucks (but allowing small vehicles) is likely sufficient. In the absence of additional data, it is recommended to consult the adhesive manufacturer and follow their guidance.

6.2 CFRP-Based Techniques

Many emerging repair technologies employ the use of Fiber Reinforced Polymer (FRP) materials. FRP materials consist of a polymer matrix reinforced with a high performance fiber. Fiber materials may be aramid (AFRP, uncommon in North American practice), carbon (CFRP), glass (GFRP), or high performance steel (SFRP) or hybrids of these. A list of applicable national and international specifications and guides pertaining to the use and design of FRP composite repairs for strengthening, repair and rehabilitation are provided in Appendix I.

6.2.1 Material Selection

For highway infrastructure applications, preformed CFRP strips are preferred over wet layed-up CFRP for flexural repair. Preformed strips are available from a variety of manufacturers in discrete sizes and a number of 'grades' of CFRP: high strength (HS), high modulus (HM) and ultra high modulus (UHM). Properties of each of these are provided in Table 3. Preformed UHM-GFRP (glass FRP) is also commercially available; however this material is relatively soft and not well suited for flexural repair. HS-CFRP is the most readily available material and most commonly used for concrete repair applications. Although greater material efficiency may be realized using the higher modulus varieties, the reduced stress and strain capacity make catastrophic CFRP rupture more likely, reducing the stress allowable in design (see Eq. 4).

Table 3 Representative properties of available preformed FRP materials.

	HS-CFRP	HM-CFRP	UHM-CFRP	UHM-GFRP
Tensile modulus, E_f (ksi)	23200	30000	44000	6100
Tensile strength, f_{fu} (ksi)	406	420	210	130
Rupture strain, ε_{fu}	0.017	0.014	0.005	0.021
Typically available strip thickness, t_f (in.)	0.047	≈ 0.05	≈ 0.05	0.075
Typically available strip widths, b_{f1} (in.)	2, 3 and 4	4	4	2 and 4

6.2.2 Environmental Durability

Based on a recent study (Cromwell et al. 2011) the reduction factors (C_E) given in Table 4 are recommended for use with exterior bridge applications [1.5.3.5]. These factors are applied to both FRP strength (i.e., $C_E f_{fu}$) and modulus ($C_E E_f$) values.

Table 4 Proposed environmental reduction factors, C_E

	CFRP strips	CFRP fabric	GFRP strips	GFRP fabric
FRP Material Properties (f_{fu} and E_f)	0.90	0.90	0.80	0.80
Bond Capacity	0.90	0.50	0.90	0.50

ACI 440-2R (2008) recommends a value of C_E = 0.85 and C_E = 0.65 for CFRP and GFRP materials properties, respectively. No reduction is used for bond capacity.

6.2.3 Fatigue

FRP materials, particularly CFRP, exhibit excellent performance when subject to fatigue loads [1.5.3.4]. In conditions of tension fatigue where environmental effects are not affecting behavior, CFRP composite behavior is dominated by the strain-limited creep-rupture process. In terms of tensile S-N behavior, CFRP material degrades at a rate approximately one half that of steel (i.e.: slope of S-N curve is half that of steel). CFRP composites generally do not exhibit a clearly defined endurance limit under conditions of tension fatigue.

The performance of externally bonded (EB) CFRP systems deteriorates when subject to fatigue loading. Fatigue along the bond interface affects the FRP strain at which debonding initiates but, in a well detailed application where debonding is controlled, will not result in a significant reduction in ultimate repaired member capacity. Ductility or deformation capacity is reduced, although this is not typically a significant concern for prestressed concrete girders. Additionally, even very low stress ranges result in some degree of degradation suggesting that there is no (or at least a very low) endurance limit below which fatigue-induced degradation is no longer a concern.

Adhesive (epoxy) selection can mitigate some effects of fatigue. For relatively low fatigue stress ranges typical of prestressed girder repairs, a stiffer adhesive will exhibit minimal degradation. At higher stress ranges, however, degradation should be expected and a softer adhesive will provide greater ductility and may be expected to behave in a more predictable manner. It is noted that in terms of efficient stress transfer under static loads, a stiff adhesive is preferred. Therefore, except in cases where fatigue effects are dominant, well designed and detailed EB-CFRP systems will perform well. Post-tensioned bonded systems (bPT-CFRP, see below) further mitigate the deleterious effects of fatigue.

6.2.3.1 Fretting Associated with Fatigue Loads

A secondary effect of fatigue loads is the relative movement that occurs between the girder substrate and unbonded repair measures (uPT-CFRP and PT-steel). In such cases, unbonded repair materials must be physically isolated from the substrate girder to avoid the possibility of fretting along this discontinuous interface. When considering fretting, anticipated girder repair system deformations should be accounted for.

6.2.4 Externally Bonded Non Post-Tensioned CFRP Retrofit (EB-CFRP)

EB-CFRP systems are the recommended technique for repairing impact-damaged prestressed girders not requiring the restoration of some prestress force (SEVERE I and II).

CFRP strips adhesively bonded to prestressed concrete girders can restore or increase the flexural capacity of damaged girders, control cracking if it is present and reduce deflections under subsequent load [1.5.3.1]. This application is shown schematically in Figure 5b.

CFRP materials for EB applications may take the form of preformed strips or wet layed-up fabrics. Strips are generally unidirectional fiber reinforced plates having a very high fiber volume ratio; thus they are axially very stiff and strong. Strips are bonded to the prepared concrete substrate using conventional structural adhesives. Wet layed-up fabrics are saturated *in situ*. Typically, a layer of epoxy resin is placed on the prepared concrete substrate as a ‘primer’ layer and the dry CFRP fabrics are overlayed onto this, an additional layer of saturant is applied and the material is worked to ensure complete epoxy wet-out of the fiber. Additional layers are applied in the same fashion. The resulting CFRP material generally has a lower stiffness and strength based on the area of the resulting CFRP plate. Wet layed-up applications are suitable for column wrapping and U-wrap applications, however are not generally recommended for flexural repair for prestressed concrete girders [1.5.3.1.1]

6.2.4.1 Limit States of EB-CFRP Repairs

The dominant limit state for bonded CFRP applications is debonding of the CFRP from the substrate concrete. It is important to note that in a sound CFRP application, debonding failure is characterized as a cohesive failure through a thin layer of cover concrete immediately adjacent the CFRP. Because of this, it is difficult to improve debonding capacity through adhesive selection (provided the adhesive is adequate to result in failure in the concrete) since the failure is governed by the substrate concrete whose properties remain unaffected in a repair. Guidance such as that provided by ACI 440-2R (2008) or NCHRP *Report 609* (2008) are intended to ensure a sound CFRP application in which the design process considers the debonding limit state.

Debonding imparts a level of pseudo ductility to the failure and is preferred to CFRP rupture. In the design of bonded FRP systems, the strain carried by the FRP is limited to a value corresponding to the strain to cause debonding, defined as (ACI 440-2R 2008):

$$\varepsilon_{fd} = 0.083\sqrt{\frac{f_c'}{nE_f t_f}} \leq 0.9\varepsilon_{fu} \quad \text{(ksi units)} \qquad \text{(Eq. 4)}$$

Where f_c' = concrete substrate compressive strength
E_f = tension modulus of elasticity of CFRP
t_f = thickness of one ply/layer CFRP
n = number of layers of CFRP
ε_{fu} = rupture strain of CFRP strip
the leading coefficient, 0.083, is taken as 0.41 for MPa units

Using preformed strips, the effectiveness of a repair is maximized by first maximizing the coverage of the CFRP on the soffit of the structure; i.e.: maximizing the width of the CFRP application, b_f. As can be seen from Eq. 4, additional layers of CFRP (n) or additional CFRP thickness (nt_f) are not proportionally effective [2.3.3.1].

Because bond capacity does not permit the full utilization of the CFRP, when small amounts of CFRP are used, these may debond prior to providing any enhancement of the *ultimate* capacity of the girder. In this case, the capacity at the *debonding limit state* is below the damaged girder capacity. In an externally bonded system, the debonding limit state is taken as being critical since once the bond fails, the girder is no longer strengthened. This hierarchy of failure also justifies the limit provided by Eq. 3.

6.2.5 External Prestressed (P-CFRP) and Post-Tensioned (PT-CFRP) CFRP Retrofit

A parallel can be drawn between prestressed and non prestressed CFRP retrofits and prestressed and conventionally reinforced concrete beams. The benefits of stressing CFRP strips prior to application include i) better utilization of the strengthening material; ii) smaller and better distributed cracks in concrete; iii) unloading (stress relief) of the steel reinforcement; resulting in iv) higher steel yielding loads. Another advantage of using P-CFRP or PT-CFRP systems is the potential for the restoration of service level displacements or performance of the structure. These systems have a confining effect on concrete (and, significantly, any patch material) because they place the concrete into compression. Therefore, a delay in the onset of cracking and a reduction of crack widths (only in bonded systems) is possible when this technique is used

Conventionally used CFRP materials have about 1.5 times the tensile capacity of 270 ksi steel prestressing strand and a Young's modulus about 75% of that of steel, meaning they can reach a higher strain. Stressing the CFRP for the repair can reintroduce prestressing force, lost due to damage or strand loss, back into the beam allowing for redistribution and a decrease of stresses in the strands and concrete. Thus when reloaded, the stress levels in the remaining strands will be reduced as compared to the unrepaired beam. In other words, prestressed CFRP systems create an active load-carrying mechanism which ensures that part of the dead load is carried by the CFRP whereas non prestressed EB-CFRP can only support loads applied after installation of the CFRP on the structure.

There are three approaches to prestressing or post-tensioning (the terms are used inconsistently in the literature) CFRP. The following terminology is adopted to clarify the types of prestressed CFRP systems. Figure 5 provides a schematic representation of each approach.

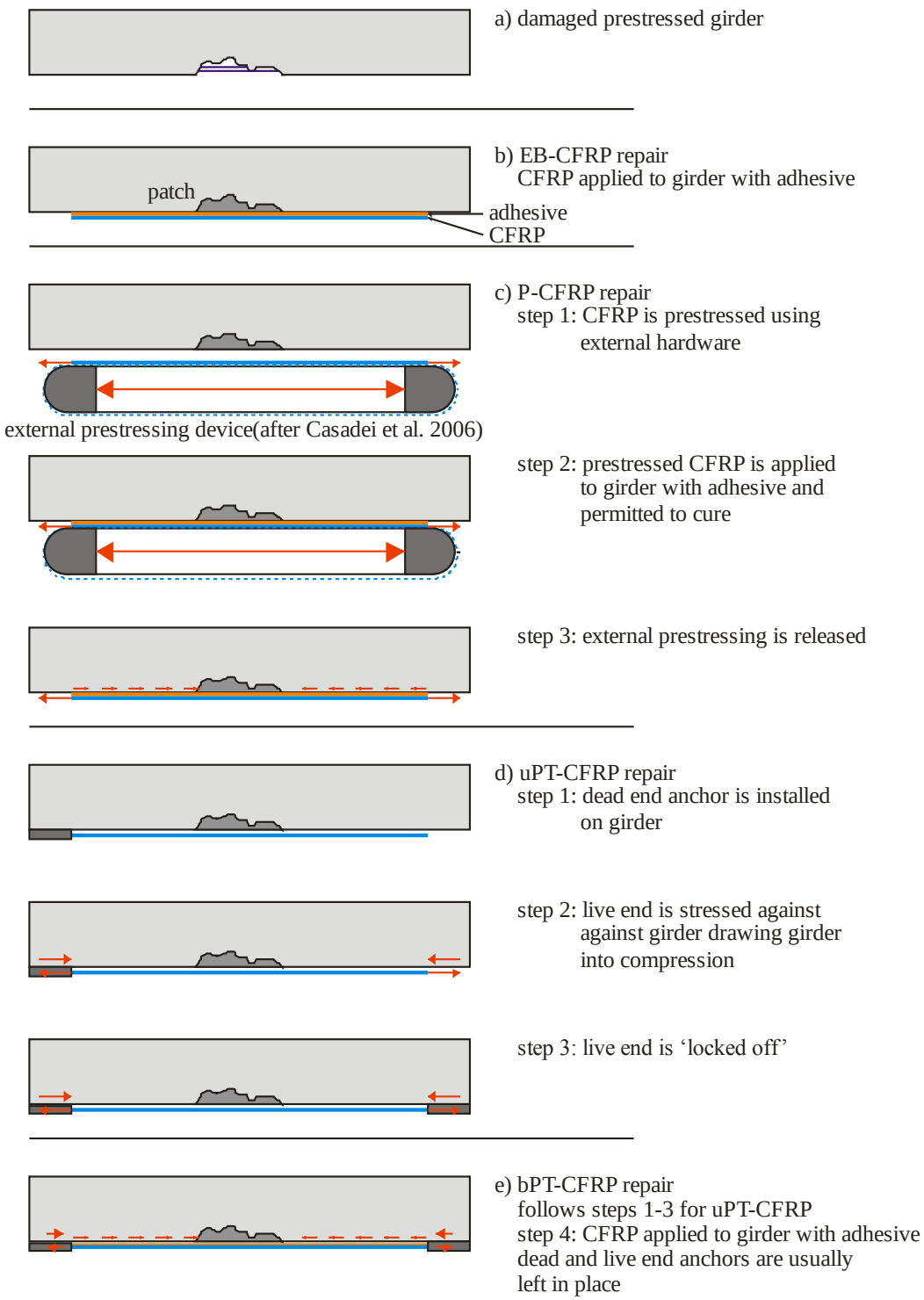


Figure 5 Schematic representations of CFRP applications.

6.2.5.1 Prestressed CFRP (P-CFRP)

Due to their only marginal improvement in performance over EB-CFRP systems and considering the complexity of their application, P-CFRP systems are not recommended as a repair method for impact-damaged prestressed concrete girders.

In a P-CFRP system, the CFRP is drawn into tension using external reaction hardware and is adhesively bonded to the concrete substrate while under stress (Figure 5c). The stress is maintained using the external reaction until the bonding adhesive is cured. The reacting stress is released and the 'prestress' is transferred to the substrate concrete. This method of prestressing is susceptible to large losses at stress transfer and long term losses due to creep of the adhesive system. As a result, only relatively low levels of prestress may be achieved. Additionally, details (such as FRP U-wraps) must be provided to mitigate debonding at the termination of the CFRP strips. P-CFRP systems are analogous to prestressed concrete systems where the stress is transferred by bond to the structural member.

6.2.5.2 Unbonded post-tensioned CFRP (uPT-CFRP)

Due to the increased susceptibility to fretting damage, uPT-CFRP are not recommended as a repair method for impact-damaged prestressed concrete girders. bPT-CFRP systems overcome the drawbacks of uPT-CFRP at little additional cost.

In a uPT-CFRP system, the CFRP is drawn into tension using the member being repaired to provide the reaction. The stress is transferred to the member by mechanical anchorage only (Figure 5d). Typically a hydraulic or mechanical stressing system will be used to apply the tension after which it will be 'locked off' at the stressing anchorage. This method of post-tensioning is susceptible to losses during the 'locking off' procedure. Depending on the anchorage method, long term losses due to creep in the anchorage are a consideration. Such systems must be designed with sufficient clearance between the CFRP and substrate concrete to mitigate the potential for fretting. uPT-CFRP systems are analogous to conventional unbonded steel post tensioning systems.

6.2.5.3 Bonded post-tensioned CFRP (bPT-CFRP)

bPT-CFRP systems are the recommended technique for repairing impact-damaged prestressed girders requiring the restoration of some prestress force (SEVERE III).

In this technique, the CFRP is stressed and anchored in the same fashion as the unbonded systems. Following anchorage, however, the CFRP is bonded to the concrete substrate resulting in a composite system with respect to loads applied following CFRP anchorage (Figure 5e). Since the adhesive system is not under stress due to the post-tension force, adhesive creep is not as significant a consideration with this system. The bonding of the CFRP may also help to mitigate creep losses associated with the anchorage. bPT-CFRP systems are analogous to conventional bonded steel post tensioning systems.

6.2.5.4 Limit States of PT-CFRP Repairs

The advantages of prestressing a CFRP repair are i) that the CFRP becomes active and replaces some of the prestress force lost due to internal strand loss; and ii) the CFRP debonding strain is effectively increased by the level of prestress in the CFRP:

$$\varepsilon_{fd} = 0.083\sqrt{\frac{f_c'}{nE_f t_f}} + \kappa\varepsilon_{fu} \le 0.9\varepsilon_{fu} \qquad \text{(ksi units)} \qquad \text{(Eq. 5)}$$

Where κ is the effective level of prestress in the CFRP and the remaining parameters are provided in reference to Eq. 4. Losses are relatively significant in PT-CFRP systems. Based on relatively limited available data, κ should remain less the 0.50 for bPT-CFRP (Figure 5e) and 0.30 for uPT-CFRP (Figure 5d) systems. Due to the stressing equipment required, prestressed CFRP (P-CFRP; shown schematically in Figure 5c) is not considered to be practical for bridge structures and is not considered further. Furthermore, for bridges or other applications subject to significant transient loads, uPT-CFRP is not recommended. uPT-CFRP strips will be located against the substrate concrete and therefore be subject to abrasion or fretting damage associated with differential displacement between the concrete substrate and unbonded CFRP strip (see Section 6.2.3.1). This situation must be remediated as the CFRP materials do

not have a great resistance to abrasion. Bonding this interface addresses this issue. For this reason, only bPT-CFRP are recommended.

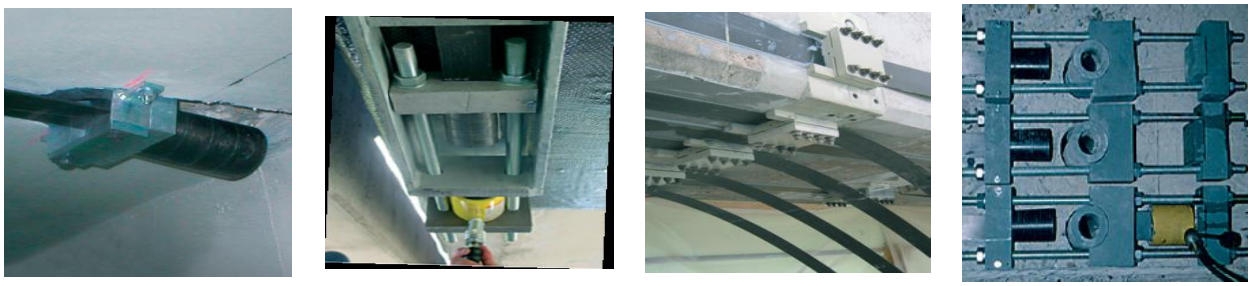
Anchorage of PT-CFRP is usually provided by proprietary anchorage hardware which in turn is anchored to the concrete substrate (see Section 6.2.5.5). The CFRP-to-anchor connections may rely on adhesive bond, friction or bearing of a preformed CFRP 'stresshead' (Figure 6a). The proprietary anchor, in turn, is secured to the concrete substrate. Anchor bolts (seen in Figure 6a) and shear keys are conventional methods of transferring the force. For anchorages bolted to the concrete substrate, the recommendations ACI 318 (2011) Appendix D for bolting to concrete should be followed and great care must be taken to ensure that installed bolts do not interfere with sound internal prestressing strand or other reinforcement. For anchorages relying on a shear key arrangement, the key should be designed to carry 100% of the prestress force and bolts should be provided to carry any uplift caused by moment and to keep the shear key fully engaged. Pipe-type shear key inserts, as are occasionally used with the system shown in Figure 6, are impractical for prestressed members in regions where strand is present. Anchorage requirements such as available space and bolt spacing may affect the amount of post-tensioned CFRP that may be installed.

Due to their size, adjacent anchorages must be staggered longitudinally if a large amount of CFRP is required. Additionally, based on commercially PT-CFRP systems (see Section 6.2.5.5), the minimum transverse spacing of the CFRP strips is typically twice the strip width. Thus the coverage of PT-CFRP on the girder soffit is $b_f \leq 0.5b$. This diminishes some of the effectiveness of PT-CFRP although some systems utilize a thicker CFRP strip than is available for EB-CFRP.

6.2.5.5 Methods of Prestressing CFRP

There are significant challenges associated with prestressing CFRP strips. CFRP materials have highly orthotropic material properties. The transverse stiffness and strength of unidirectional CFRP strips and fabric systems may be orders of magnitude less than the longitudinal properties that make these materials good alternatives for prestressing in the first place. This makes CFRP materials difficult to 'grip' in order prestress. A variety of solutions have been demonstrated in laboratory and 'pilot' applications [1.5.3.2.1 and 1.5.3.2.2] although few are practical for full scale deployment.

There are only two known commercially available 'standardized' PT-CFRP systems. Both systems work on the same principle, have essentially the same capacities and differ only in the proprietary nature of the jacking hardware used. An example of such a system is shown in Figure 6. The anchorage has a capacity of 67 kips (300 kN) and is intended for a maximum applied prestress force of 45 kips (200 kN). Material properties of the CFRP strips are those given for HS-CFFP in Table 2. This system is comprised of CFRP strips with 'potted' CFRP anchorages referred to as 'stressheads' manufactured on each end. These stressheads are captured by steel anchorages mounted on the concrete (Figure 6a) or by the jacking hardware (Figures 6b and d). One anchor is the fixed or 'dead' end (Figure 6a) while the other is the jacking end (Figure 6b). The jacking end stresshead connects into a movable steel frame which connects to a hydraulic jack, thus allowing the strip to be stressed. Once the desired stress level is reached, the jack can be mechanically locked to retain the stress in the CFRP or the CFRP strip can be anchored by 'clamps' (Figure 6c) near the jacking end. Anchor points can also be located at discrete intervals along the beam. The stress in the strips can vary according to structural need and is limited to the tensile strength of the strip. In many cases, the strength of the beam at the anchor location controls the amount of prestress force that can be applied.



(a) dead end anchor (b) jacking end anchor in movable frame (c) multiple live end anchors at one location (d) stress head system

Figure 6 Components of commercially available PT-CFRP system.

6.2.6 Near Surface Mounted CFRP reinforcement

Near-surface mounted (NSM) CFRP repairs provide an alternative to externally bonded repairs. The NSM technique places the CFRP in the cover concrete of the member (see Figure 7). This protects the material from impact forces and environmental exposure. Although there is no prohibition against prestressing NSM CFRP, there is presently no practical method by which to prestress such systems. An NSM CFRP repair is completely enclosed in epoxy, making it possible to achieve higher bond strength as compared to external strip bonding due to the larger surface area which is bonded. Additionally, an NSM application engages more cover concrete and is able to transfer greater stresses into the concrete substrate. Since bond capacity is typically the limit state associated with EB repairs, NSM repairs will typically require less CFRP material due to the enhanced bond characteristics. NSM repairs are sensitive to the amount of concrete cover and are not a viable option when cover is not sufficient.

NSM repairs are not recommended as being efficient for repairs in the positive bending region of a structure (see 6.2.6.1). The significant advantage of NSM techniques is that they may be used in the negative moment region of a structure while remaining protected from wear and abrasion. Negative moment region repairs are not an application considered within the scope of this *Guide*.

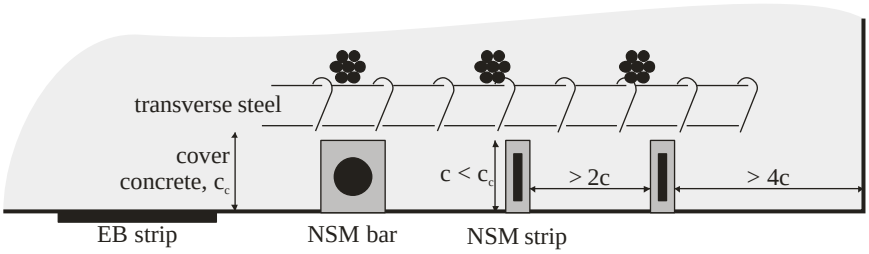


Figure 7 Schematic of externally bonded and NSM CFRP techniques.

6.2.6.1 Limit States of NSM-CFRP Repairs

For near-surface mounted applications, bond is improved over EB applications. ACI 440-2R (2008) prescribes a debonding strain of $\varepsilon_{fd} = 0.7\varepsilon_{fu}$. However, the improved bond effectiveness is often negated by the limited amount of material that may be applied in this manner: Slots for NSM reinforcing should be spaced at least twice their depth apart and have an edge distance of at least four times their depth as shown in Figure 7 (ACI 440-2R 2008). Slots must not encroach on the internal reinforcing steel; for typical prestressed concrete structures, slots will not exceed 1 in., and may be better specified to be 0.75 in. in depth. The CFRP inserted into the slot is necessarily shorter than the slot depth although typically two plies of preformed CFRP will be used in each slot. For such typical prestressed concrete applications, NSM-CFRP will typically consist of 2-0.5 in. strips inserted into slots spaced at 1.75 in. This provides 50% of the material that an EB-CFRP application provides at a greater cost in terms of resources and

installation time. The improved bond behavior will normally not improve the CFRP efficiency the required 200% just to 'break even' in terms of capacity.

6.2.7 Maximum Effect of CFRP Repair Techniques

Although repair designs must be considered on a case-by-case basis, it is useful to have an approximate measure of the effectiveness of the repair material in restoring moment capacity and to relate this to the *in situ* girder being repaired [2.6.1]. By equating the nominal moment capacity contribution of the CFRP to an equivalent contribution of the *in situ* prestressing strand, one arrives at the theoretical maximum number of severed prestressing strands, n_{max}, that can be replaced by CFRP based on its relative contribution to moment capacity:

$$n_{max} = \frac{E_f n t_f b_f \varepsilon_{fd} \alpha H}{f_{pu} A_p \beta H} \qquad \text{(Eq. 6)}$$

Where: E_f = tension modulus of elasticity of CFRP

t_f = thickness of one ply/layer of CFRP

n = number of plies/layers of CFRP

EB-CFRP: for preformed strip it is recommended that n = 1

PT-CFRP: n = 1

b_f = maximum width available for CFRP bonding

EB-CFRP: $b_f = b$

PT-CFRP: $b_f = 0.5b$

NSM-CFRP: $b_f = (b/2 + 1)c$

b = soffit width of girder; if chamfers are present, the available soffit width should be reduced accordingly

c = depth of NSM slot

ε_{fd} = debonding strain of CFRP:

EB-CFRP: $$\varepsilon_{fd} = 0.083\sqrt{\frac{f_c'}{nE_f t_f}} \le 0.9\varepsilon_{fu} \quad \text{(ksi units)} \qquad \text{(Eq. 4)}$$

PT-CFRP: $$\varepsilon_{fd} = 0.083\sqrt{\frac{f_c'}{nE_f t_f}} + \kappa\varepsilon_{fu} \le 0.9\varepsilon_{fu} \quad \text{(ksi units)} \qquad \text{(Eq. 5)}$$

NSM-CFRP: $\varepsilon_{fd} \le 0.7\varepsilon_{fu}$

ε_{fu} = rupture strain of CFRP strip

f_c' = concrete substrate compressive strength

$\kappa = 0.3$ for uPT-CFRP

$\kappa = 0.5$ for bPT-CFRP

αH = depth from compression resultant to location of CFRP on soffit

βH = depth from compression resultant to centroid of prestressing strands

H = overall depth of girder

f_{pu} = ultimate stress of prestressing strand

A_p = area of one prestressing strand

Values of α and β vary based on the section geometry and amount of prestressing provided. In lieu of member-specific calculations, the values given in Table 5 may be used as estimates of these values. The ratio α/β effectively normalizes the CFRP stress to that of the *in situ* strand accounting for section geometry. The CFRP, located at the extreme tension fiber, is more efficient at providing moment capacity to the section than the *in situ* strand due to its greater level arm to the compression resultant, αH.

Table 5 Approximate values of α and β.

	rectangular section	voided slab	I-girders	bulb tees	deck bulb tees	box girders
α	0.65	0.65	0.80	0.80	0.90	0.92
β[1]	0.50	0.58	0.64	0.64	0.76	0.82

[1] Collins and Mitchell (1997)

6.2.7.1 Restoration of Prestressing Force

In addition to restoring the ultimate load carrying capacity of a member, PT-CFRP systems are able to restore some degree of prestress force lost along with severed strands. Using the same approach as described by Eq. 6, Eq. 7 provides an approximation for the maximum amount of prestress force that can be replaced by PT-CFRP systems. Equation 7 is normalized by the effective prestress force provided by a single strand ($f_{pe}A_p$) resulting in the equivalent number of strands whose prestress force can be replaced with PT-CFRP, $n_{max\text{-}PT}$:

$$n_{max-PT} = \frac{E_f n t_f b_f \kappa \varepsilon_{fu} \beta}{f_{pe} A_p \alpha} \qquad \text{(Eq. 7)}$$

Where f_{pe} is the long term effective prestress force; in lieu of calculating this value, $f_{pe} = 0.57f_{pu}$ may be used as a reasonable estimate for the long term effective prestress after all losses (based on *AASHTO LRFD* Section 5.9.5.3). All other values are the same as those given for Equation 6. The ratio β/α effectively normalizes the CFRP stress to that of the *in situ* strand accounting for section geometry. The CFRP, located at the extreme tension fiber, is less efficient at providing prestress to the section than the *in situ* strand.

PT-CFRP is less efficient at restoring lost prestress force ($n_{max\text{-}PT}$) than lost ultimate capacity (n_{max}). Because of the limited long term prestress force available in an uPT-CFRP system ($0.30f_{fu}$), such systems perform only marginally better than EB-CFRP. Like NSM-CFRP, it is unlikely that the additional effort required for uPT-CFRP would warrant its use. The prestress force available for a P-CFRP system is even less than an uPT-CFRP and therefore possibly less efficient than an EB-CFRP system due to geometric constraints.

6.3 Steel-Based Techniques

6.3.1 PT-Steel

External steel post-tensioning (PT-steel) is done using steel rods, strands or bars anchored by corbels or brackets (typically referred to as 'bolsters') which are cast or mounted onto the girder; typically on the girder's side (although occasionally on the soffit) as shown in Fig. 4. The high strength steel rods, strands or bars are then tensioned by jacking against the bolster (*Report 280*). Design of PT-steel repair systems are well established using simple plane sections analysis (recognizing that the post-tensioning bar is unbonded). Any degree of damage may be repaired and prestress force restored using this method. The primary design consideration is the transfer the PT force into the girder through the bolsters. This is accomplished through design of the bolster as a bracket or corbel (see *AASHTO LRFD Specifications* Section 5.13.2.4) or through direct bearing of a shear key. Typically, the bolsters themselves will be post-tensioned onto the girder web thereby affecting a normal force (P_c in AASHTO LRFD Equation 5.8.4.1-3) anchoring the bolster. When adding bolsters to box girders, the web thickness (and possible variation thereof) must be considered: Ideally, the bolsters are post-tensioned to a single web although this will likely require access inside the box. If post-tensioning the bolster through an entire box girder, this should be done at the location of an internal diaphragm in order not to affect out-of-plane bending of the web. In general, applying PT-steel repair methods to box sections is perhaps not as practical as to single web members. PT-steel may also be anchored to existing or purpose-cast diaphragms between girders as shown in Figure 4b.

Generally it will be more efficient to harp external post tensioning. This is easily done, although the attachment of the harping points to the girder requires the same attention as the end anchorages. Additionally, harping points damaged in a subsequent impact may result in a catastrophic failure; therefore harping points should not be located in impact-susceptible areas. Finally, external PT-steel must be adequately protected from corrosion; grease-filled ducts/tubes are recommended. Epoxy-coated strand used for external PT-steel repairs should be provided with additional protection against corrosion.

External post-tensioning may also use CFRP cables (CFCC). These systems have proprietary anchorages that interface with conventional prestressing hardware. Such systems are rare in North American practice.

6.3.2 Steel Jackets

Due to their complexity and the fact that they are untested, steel jacket repairs are not recommended as a repair method for impact-damaged prestressed concrete girders; it is believed that CFRP repairs address all advantages of steel jackets while overcoming some of their drawbacks.

Steel jacketing is the use of steel plates to encase the girder section to restore girder strength. Generally, this method of repair will also require shear heads, studs or through bars to affect shear transfer between the steel jacket and substrate girder. Steel jacketing is felt to be a very cumbersome technique. In most applications, field welds will be necessary to 'close' the jacket (since the jacket cannot be 'slipped over' beam ends in most applications). Additionally, the jacket will need to be grouted in order to make up for dimensional discrepancies along the girder length. *Report 280* discusses steel jacket repairs.

6.4 Strand-Splicing

Strand splices are designed to reconnect severed strands. Methods of reintroducing prestress force into the spliced strand are preloading, strand heating and torqueing the splice (*Report 280*); the latter is analogous to a turnbuckle. Preloading to develop prestress forces in a strand splice will generally be impractical (see Section 6.6) and strand heating is not recommended.

Strand splicing is well established and an effective means of restoring steel continuity and prestress force to severed strands. Commercially available strand splices have couplers connected to reverse threaded anchors; as the coupler is turned, both anchors are drawn toward each other, introducing a prestress in the attached strand (see Figure 8). Commercially available strand splices are reportedly adequate to develop $0.96f_{pu}$. Typically, a re-tensioning operation will aim to restore $0.60f_{pu}$ which will generally be close to the long-term effective prestress in a strand. Commercially available splices are available for strand diameters only up to 0.5 in. [2.3.3.4].

6.4.1 Limit States of Strand Splice Repairs

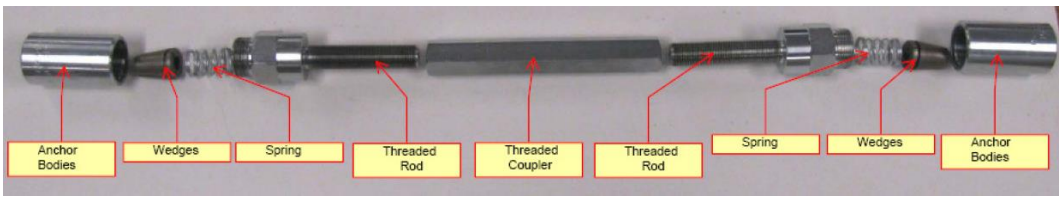
Limitations pertaining to the use of strand splices are based on the physical dimensions of the splices and spacing of the strands to be spliced. Often, prestressing strands are spaced at 1.5 in. on center. Anchor bodies (Figure 8a) have a diameter of 1.625 in. irrespective of spliced strand size. When repairing adjacent strands, splices must be staggered (Figure 8b) to avoid interference (Figure 8c). The specified stagger must accommodate the 'stroke' of the splice coupler; a minimum spacing of 2 in. is recommended (Figure 8b). Installation of splices for exterior strands also results in reduced concrete cover at the splice location unless section enlargement is also affected. Reduced cover may affect durability and result in a region more susceptible to cracking. Clear spacing between splices and adjacent bonded strands is also reduced (Figure 8c) which may affect bond performance and increase the development length of adjacent intact strand.

Chuck splices, having a diameter of 2 in. are used to allow for removal of damaged strand and to shift the location of the splice to accommodate staggering and reduce congestion (Figure 8b). Chuck splices must also be staggered and not coincide with strand splice locations in order to avoid interferences. Staggering strand splices often requires the removal of additional strand and its surrounding concrete. Finally, strand splices and chuck splices also interfere with transverse reinforcement where it is present. This may

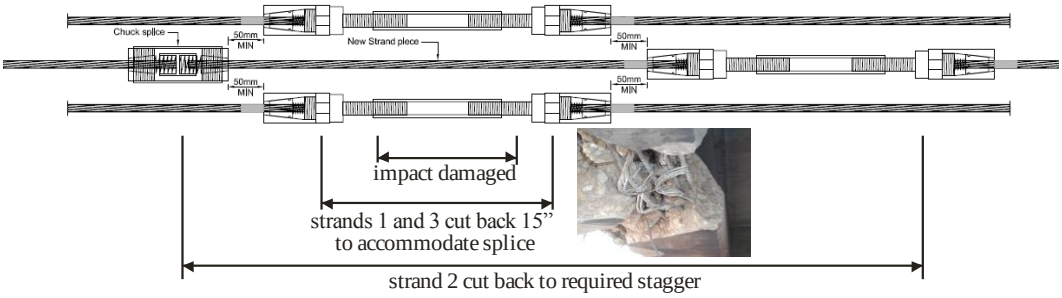
require removal of the transverse steel and replacement with grouted or epoxied hairpins. In some cases, FRP U-wraps (see Section 7.2) may be designed to restore the confinement and shear capacity lost by the removal of transverse steel (ACI 440-2R 2008).

Considering interferences, strand splicing is practical for relatively few severed strands but becomes increasingly cumbersome for significant damage – particularly for the case of adjacent damaged strands. For this reason, a combination of external repair and strand splicing (see Section 6.5) may be appropriate where a large number of strands are damaged, particularly if this damage is confined to a short length of the girder.

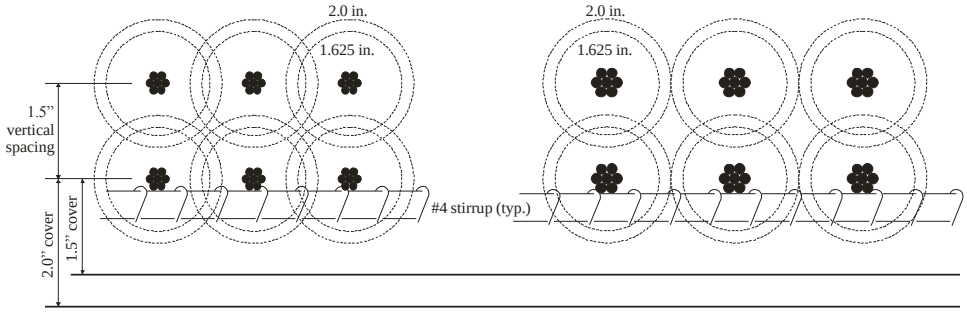
Strand splicing requires sound strands. Any corroded region of strand must be removed back to 'bright steel'. Thus if impact damage has been left unrepaired and corrosion set in (see Section 1.2.1), strand splicing may require additional strand and concrete removal.



a) components of strand splice [Prestress Supply Inc. 2011]



b) required longitudinal stagger of adjacent strand splices [Prestress Supply Inc. 2011]



3/8" strand at 1.5 in. spacing 1/2" strand at 2.0 in. spacing

c) strand splice interferences

Figure 8 Strand splice hardware and potential interferences.

6.4.1.1 Strand Splice Ultimate Capacity

Strand splices cannot be universally relied upon to develop the ultimate capacity of the strands. Some research indicates that 100% of capacity can be restored while others report that only about 80% of

capacity can be restored. Zobel and Jirsa (1998) report a 'guaranteed' strength of $0.85f_{pu}$ while the available commercial literature implies a capacity of $0.96f_{pu}$ for strand splices. Zobel and Jirsa also recommend that more than 10-15% of strands in a section be spliced. Thus it is assumed that strand splices are able to restore *in situ* prestress forces assumed to be less than $0.70f_{pu}$. Furthermore, it is recommended that strand splices:

- be limited to strand diameters 0.5 in. and less
- be limited to developing $0.85f_{pu}$, thereby reducing their effectiveness in restoring strands. The effective number of strands restored by strand splices should be taken as $0.85n_{spliced}$
- be staggered (Figure 8b) when splicing adjacent strands.
- be limited to splicing 15% of strands in a girder regardless of staggering.

The Alberta Infrastructure and Transportation Department (ABITD 2005) provides additional guidance with respect to the evaluation and acceptance criteria of- and procedure for restressing severed prestressing strands. In particular, the need to calibrate the restressing operation is emphasized and guidance provided [1.5.1.2].

6.4.1.2 Strand Splicing in Box Girders

Beyond the repair of a few isolated strands, it is not recommended to use strand splicing for impact-damaged box girders, particularly older girders. Anecdotal evidence has indicated significantly reduced cover concrete (as small as 0.75 in. to the center of the strand) and inconsistent strand spacing (as small as 1 in. center-to-center) in girders from the 1960's and early 1970's. In any event, splicing adjacent strands in a box girder will result in significantly reduced concrete cover and interferences. Issues of splitting associated with strand splices cannot be adequately addressed in box girders since the top of the bottom flange (in the box void) is inaccessible. Providing new internal confinement (grouted hairpins) is impractical since these cannot be anchored in the 4 to 5 in. bottom flanges. External confinement (such as CFRP U-wraps) is also not as efficient since this confinement cannot be provided inside the box void. Section enlargement ('blisters') of box girders may be used but is not as practical as for flanged members and reduces the roadway clearance below the bridge.

6.4.1.3 Strand Splicing in Single-web Girders

Strand splicing is more practically applied to 'flanged' members. The reduced cover, spacing and interferences with adjacent strands/splices associated with strand splices affect development, crack control and the likelihood of splitting. For this reason, confinement of the spliced region is recommended to ensure long-term durability and performance. In flanged sections such as AASHTO I-girders and bulb tees, section enlargement, grouted or epoxied hairpins and/or CFRP (or GFRP) U-wraps may be used to confine the spliced region, control splitting cracks and replace any transverse reinforcement that may have been removed to affect the splice repair (ACI 440-2R 2008).

Splicing adjacent strands requires a longer repair region (Figure 8) and may therefore not be practical for cases of very local damage (such as is shown in the inset in Figure 8). In this case a hybrid approach may be efficient: repairing some strands (perhaps every second strand, limited to 15% of the strands in the affected section) with strand splices and restoring the remaining capacity with an externally bonded alternative. This approach is described in the following section.

6.5 Hybrid Repairs

Limitations associated with each repair technique clearly point to the adoption of hybrid repair approaches in order to maximize the degree of damage that may be repaired.

Strand splices are internal applications and therefore may be used with most any external application (except, perhaps NSM where interference between the strand chucks and NSM slots is likely). Strand splices are reported to have the capacity to restore *in situ* prestress to all but 0.6 in. strands. Typically, *in situ* levels of long term prestress will be on the order of $0.60f_{pu}$. As reported in Section 6.4.1.1, strand

splices cannot be relied upon to develop the ultimate capacity of the strands. Thus when considering the ultimate capacity of a section, the effectiveness of strand splicing should be reduced such that the equivalent number of strands considered in calculating the girder capacity is $0.85n_{spliced}$. Using this approach, the damage that must repaired using external techniques is reduced. The use of strand splices may effectively extend the utility of other repair techniques. Additionally, since strand splices are active, they may be used to restore a degree of prestressing, avoiding decompression [2.6.1.2] and permitting passive external repair techniques to be utilized.

Hybrid combinations of techniques other than strand splices will generally be inefficient since the repair stiffnesses and capacities differ resulting in multiple systems behaving in series rather than in parallel.

6.6 Preloading Structural Repairs

Preload is the temporary application of a vertical load to the girder during repair [1.5.1.1]. The preload is provided by either vertical jacking or, more conventionally a loaded vehicle. If the damage has caused a loss of concrete without severing strands, preloading during concrete restoration can restore capacity to the girder without adding prestress. In this case, preloading may be used to restore partial or full prestress to the repaired area; effectively reducing tension in the repaired area during live load applications. Preloading is most effective for smaller prestressed girders. As elements become larger the level of preload required becomes very large and often impractical to apply. The effectiveness of preload is improved with reduced dead-to-live load ratios; however these are not typical in concrete bridge structures [1.5.4.1.1].

It is believed that preloading in order to affect or restore prestress force, while feasible, is typically impractical. Preloading in order precompress patch repairs, however, is recommended (see 7.1.7).

Care should be taken when preloading a structure so as to not overload the structure or cause damage from excessive localized stresses from the preloading force.

7 PATCHING AND CORROSION MITIGATION

7.1 Patching

Concrete patching repairs are typically necessary to address defects that occur during fabrication, shipping, handling or erection of a member but can also be used to correct impact damage. Aesthetic repair methods are also often required as a final step in a structural repair. Sound patches are particularly important when they serve as substrates for bonded external repair measures (EB-CFRP and bPT-CFRP) and especially so if an NSM-CFRP repair crosses a patched area. Patching methods are well established and extensive guidance on their application are reported in the *Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products* (PCI MNL-137-06) published by the Precast/Prestressed Concrete Institute (PCI). The ICRI/ACI *Concrete Repair Manual* also provides guidance on concrete aesthetic repairs.

Patching requires the removal of all unsound concrete. It is usually a good idea to remove slightly more concrete rather than too little, unless it affects the bond of prestressed strands. The chipped area for patching should at least be 1 in. deep and should have edges as straight as possible, at right angles to the surface. The use of air driven chipping guns or a portable power saw for cutting concrete is recommended for removal of concrete but care should be taken to make sure that the reinforcement or the strands are not damaged. When patching prestressed elements, preloading the elements is often recommended in order to slightly ‘prestress’ the patch to resist ‘pop-out’; this is not always possible in *in situ* repairs.

The selection of a patching technique and material will depend on the size of the patch and limitations of each method. Considerations in patch material selection include i) rheology of the patch material (the material must thoroughly fill or pack into the void being patched); ii) bond strength to *in situ* concrete and steel reinforcement traversing patch; iii) compressive and tensile strength of patch material; and, iv)

durability of the patch material. Given that the volume of many patches is small, the benefit of using a high quality prebagged repair material is likely warranted and represents only a small incremental cost in the entire patching operation. The following six methods of patching are described and details of their application are found in *PCI Manual 137* (2006):

7.1.1 Drypack Method

The drypack method is suitable for holes having a depth nearly equal to the smallest dimension of the section, such as core or bolt holes. The method should not be used on shallow surfaces or for filling a hole that extends entirely through the section or member [1.5.5.1.1].

7.1.2 Mortar Patch Method

Mortar patches are used in concrete members with shallow defects, which require a thin layer of patching material such as in honeycombs, surface voids or areas where concrete has been pulled away with the formwork [1.5.5.1.2].

7.1.3 Concrete Replacement Method

The concrete replacement method consists of replacing the defective concrete with machine-mixed concrete that will become integral with the base concrete. Concrete replacement is preferred when there is a void extending entirely through the section, or if the defect goes beyond the reinforcement layer, or in general if the volume is large [1.5.5.1.3]. Concrete replacement will also provide the best substrate for eventual EB-CFRP, bPT-CFRP or even NSM-CFRP repairs.

7.1.4 Synthetic Patching

There are cases where Portland cement patches are difficult or impractical to apply. These situations include patching at freezing temperatures or patching very shallow surface defects. Two synthetic materials useful under such circumstances are epoxy and latex based products [1.5.5.1.4]. Epoxies can be used as a bonding agent, a binder for patching mortar, an adhesive for replacing large broken pieces, or as a crack repair material. Small deep holes can be patched with low-viscosity epoxy and sand whereas shallower patches require higher viscosity epoxy and are more expensive. It is suggested that epoxy mortars be used only in situations where exceptional durability and strength are required. Although they offer excellent bond and rapid strength development, epoxies are hard to finish and usually result in a color difference between the patch and the base concrete, clearly showing the repaired section, unless precautions (such as tinting) are taken. Latex materials are used in mortar to increase its tensile strength, decrease its shrinkage and improve its bond to the base concrete, thus helping to avoid patch failure due to differential shrinkage of the patch. Because of its good bonding qualities, latex is especially useful in situations where feathered edges cannot be avoided.

7.1.5 Prepackaged Patching Compounds

There are many commercial patching products available. A good patching material should have an initial setting time of 10 – 15 minutes and a final setting time of 20 – 45 minutes. Because some compounds generate excessive heat (leading to shrinkage and poor durability), they often are not suitable for general purpose patching [1.5.5.1.5].

7.1.6 Epoxy Injection

Epoxy injection methods have been used to repair cracks or fill honeycombed areas of moderate size and depth [1.5.5.1.6]. Only appropriately trained personnel should carry out such repairs.

7.1.7 Patch Preloading

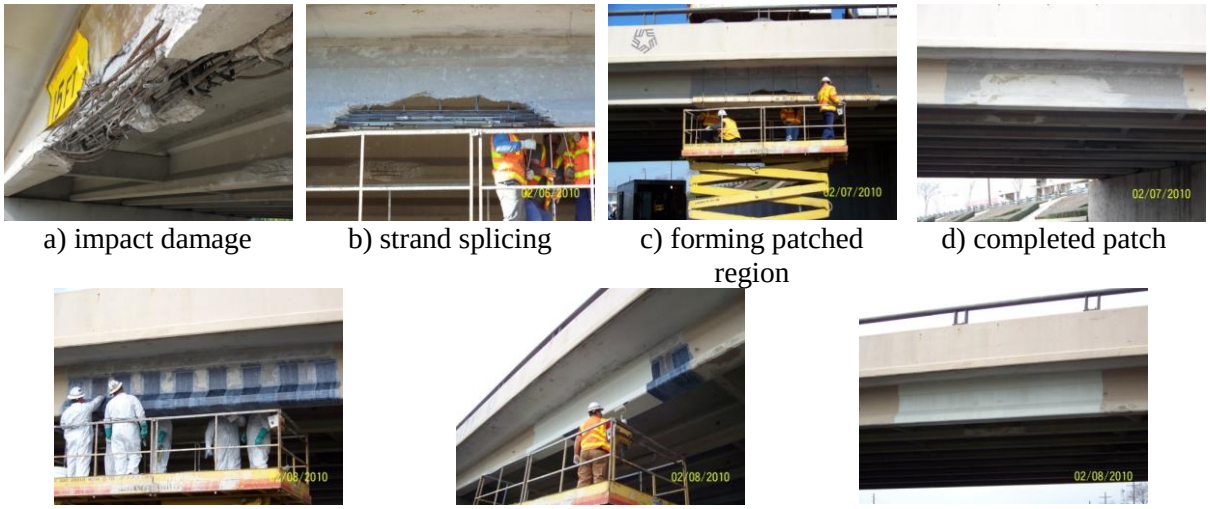
Preloading (see Section 6.6) is recommended for patch operations. Preloading structures during the patching operation can draw the patch material into compression thereby mitigating effects of patch

shrinkage, cracking and pop-out. For patching operations, preload is often provided by a vehicle parked above the patch location.

7.2 Patch Confinement – FRP U-Wraps

Preload is often impractical. As an alternative, both CFRP and GFRP materials in a 'U-wrap' fashion may be used to confine patches and resist 'pop-out' failure of the patch and shrinkage and flexural cracking in the patch [1.5.5.2]. An example repair sequence involving strand splicing, patching and patch confinement is shown in Figure 9. The externally applied FRP affords some protection to the patch and, significantly, provides some continuity or 'bridging' between the patch and surrounding concrete. Because all GFRP systems and most epoxy resin systems are susceptible to damage from UV light exposure, FRP systems applied to exterior girders in particular require a final protective top coat. This is also often done for aesthetic reasons. FRP applications used to confine patches have been found to be very robust, even serving to reduce damage caused by subsequent vehicle impacts [1.5.4.6].

In addition to the patch confinement, U-wrapped CFRP strips may be used to help 'hold' the longitudinal CFRP and improve the resistance of EB-CFRP and bPT-CFRP debonding from the concrete [1.5.3.2.2].



a) impact damage b) strand splicing c) forming patched region d) completed patch

e) CFRP confinement of patch f) protective coating for patch g) completed repair

Figure 9 Strand splicing and patch repair sequence.

[Courtesy of the Texas Department of Transportation, © 2007 All rights reserved.]

7.3 Corrosion Mitigation

When considering the repair of corroded strand, it is important to mitigate the source of corrosion prior to patching and repair. Prestressing strand is more susceptible to corrosion than lower grades of steel (due both to the composition of prestressing steel and the increased surface area-to-cross section area ratio of a seven wire strand), therefore prestressed concrete beams are susceptible to corrosion, especially at beam ends. Surface treatments and coatings are effective at mitigating corrosion in the short term, but are only effective in the long term if applied prior to chloride contamination [1.5.1.4]. The addition of FRP wraps has also shown to effectively mitigate corrosion by excluding chloride bearing water [1.5.1.4].

Active cathodic protection (impressed current systems) is also effective at mitigating corrosion, but are not commonly used due to high maintenance and monitoring costs and method complexity. Additionally, prestressing steel is particularly susceptible to cracking due to hydrogen embrittlement. If an active cathodic protection system is installed and operated in such a way that the magnitude of polarization is excessive, then atomic hydrogen may be generated at the surface of the steel and embrittlement may occur.

Passive cathodic protection (embedded anodes) installed in concrete patches is an effective method of mitigating corrosion in prestressed concrete members (Vector 2012).

8 SUMMARY OF FACTORS AFFECTING SELECTION OF PRESTRESSED GIRDER REPAIR TECHNIQUES

Ultimately, repair selection must be made on a case-by-case basis. The following matrix summarizes the utility and viability of each repair method described.

Repair Selection Criteria – EB-CFRP Techniques.

Selection Criteria	**EB-CFRP strips**		**EB-CFRP fabric[1]**		**NSM-CFRP[2]**	
primary reference	this report		this report		this report	
commercially available?[7]	yes		yes		yes	
girder type	box girder	I-girder	box girder	I-girder	box girder	I-girder
generally recommended?	yes	yes	no	yes, if bulb to be wrapped	no	no
dominant repair limit state	CFRP bond		CFRP bond		NSM slot geometry and CFRP debonding	
damage that may be repaired	Severe I and II		Severe I		Severe I and II	
prestress steel that may be replaced	≤20% of strands	≤10% of strands	≤20% of strands	≤10% of strands	≤20% of strands	≤10% of strands
active or passive?[8]	passive		passive		passive	
behavior at ultimate load	good		fair		good	
resistance to overload	limited by bond		limited by bond		good	
fatigue performance	limited by bond[9]		limited by bond[9]		uncertain	
strengthening beyond undamaged capacity?	yes		yes		yes	
combining splice methods	possible with strand splicing		possible with strand splicing		unlikely	
preload for repair[10]	no		no		no	
FRP U-wrap[10]	not feasible	recommended[11]	not feasible	recommended[11]	not required	
restore loss of concrete	patch prior to repair		patch prior to repair		patch prior to repair	
preload for patch[10]	possibly		possibly		yes	
speed of mobilization	fast		fast		moderate	
constructability	easy		easy		difficult	moderate
specialized labor required[13]	no		yes		no	
proprietary tools required	no		yes, saturation bath		no	
lift equipment required[14]	no		no		perhaps	
closure below bridge	single lane possible		single lane possible		full carriageway closure	
time for typical repair	1-2 days		2-4 days		2-4 days	
environmental impact of repair process	dust from surface preparation		VOCs from saturant and dust from surface preparation		dust from concrete sawing	
durability	requires environmental protection		requires environmental protection		excellent	
cost	low		low		moderate	
aesthetics	good		good		excellent	
retain capacity in event of subsequent impact	good		very good		very good	

Repair Selection Criteria – PT-CFRP Techniques

Selection Criteria	P-CFRP[3]		bPT CFRP		uPT-CFRP	
primary reference	this report		this report		this report	
commercially available?[7]	no		yes		yes	
girder type	box girder	I-girder	box girder	I-girder	box girder	I-girder
generally recommended?	no	no	yes	yes	no	no
dominant repair limit state	CFRP bond		anchorage design		anchorage design	
damage that may be repaired	Severe II		Severe II and III		Severe II and III	
prestress steel that may be replaced	≤20% of strands	≤10% of strands	≤40% of strands	≤20% of strands	≤20% of strands	≤10% of strands
active or passive?[8]	marginally active		active		active	
behavior at ultimate load	very good		very good		very good	
resistance to overload	limited by bond		good		good	
fatigue performance	limited by bond[9]		limited by bond[9]		excellent if fretting is mitigated	
strengthening beyond undamaged capacity?	yes		yes		yes	
combining splice methods	possible with strand splicing		possible with strand splicing		possible with strand splicing	
preload for repair[10]	no		no		no	
FRP U-wrap[10]	recommended[11]		recommended[11]		not feasible	
restore loss of concrete	patch prior to repair		patch prior to repair		patch prior to repair	
preload for patch[10]	not required		not required		not required	
speed of mobilization	moderate		moderate		moderate	
constructability	difficult		difficult		difficult	
specialized labor required[13]	yes		yes		yes	
proprietary tools required	yes, prestressing hardware		yes, PT anchors and hardware		yes, PT anchors and hardware	
lift equipment required[14]	yes		no		no	
closure below bridge	full carriageway closure		full carriageway closure		full carriageway closure	
time for typical repair	1-2 days		1 week		1 week	
environmental impact of repair process	dust from surface preparation		dust from surface preparation		minimal	
durability	requires environmental protection		requires environmental protection		requires environmental protection and protection from fretting	
cost	moderate		moderate		moderate	
aesthetics	good		fair		fair	
retain capacity in event of subsequent impact	fair		good		none if PT impacted	

Repair selection criteria – Steel-based Techniques

Selection Criteria	PT-steel		Strand Splicing		Steel Jacket[4]		Replace Girder
primary reference	*NCHRP Report 280*		this report and *NCHRP Report 280*		*NCHRP Report 280*		*LRFD Specification*
commercially available?[7]	yes		yes		no		yes
girder type	box girder	I-girder	box girder	I-girder	box girder	I-girder	all
generally recommended?	no[5]	yes	no[6]	yes	no	no[4]	yes
dominant repair limit state	anchorage design		geometry of strands to be spliced		geometry of girder		new design
damage that may be repaired	Severe II and III		Moderate	Severe I	Severe II		Severe IV
prestress steel that may be replaced	unlimited provided sufficient reserve capacity[15]		no more than 15% of strands to $0.85f_{pu}$		uncertain		unlimited
active or passive?[8]	active		active (can be installed passively)		passive		-
behavior at ultimate load	excellent		good but limited to $0.85f_{pu}$		uncertain		excellent
resistance to overload	excellent		poor		uncertain		excellent
fatigue performance	excellent		uncertain, thought to be poor		uncertain		excellent
strengthening beyond undamaged capacity?	yes		no		yes		-
combining splice methods	possible with any method		possible with any method		unlikely		-
preload for repair[10]	no		possibly		possibly		-
FRP U-wrap[10]	not required		not feasible	recommended[12]	not required		-
restore loss of concrete	patch prior to repair		patch following repair		patch prior to repair		-
preload for patch[10]	not required		yes		not required		-
speed of mobilization	moderate		fast		slow		very slow
constructability	difficult	moderate	difficult	moderate	very difficult		difficult
specialized labor required[13]	no		no		no		no
proprietary tools required	yes, PT jacks		torque wrench		maybe		-
lift equipment required[14]	yes		no		yes		yes
closure below bridge	full carriageway closure		single lane possible		full		full
time for typical repair	1 week		1-2 days		weeks		months
environmental impact of repair process	minimal		dust from concrete chipping		welding		typical erection issues
durability	requires corrosion protection		excellent		requires corrosion protection		excellent
cost	moderate		very low		moderate		high
aesthetics	fair		good	excellent	poor		excellent
retain capacity in event of subsequent impact	none if PT impacted		very good		excellent		-

Notes for Repair Selection Matrix:

1. Preformed strip CFRP materials will always out-perform fabric materials unless the repair must conform to an irregular geometry.
2. NSM repairs will generally only be used for repairs where abrasion resistance is an active concern; this will typically not be the case for impact damage repair.
3. Prestressed CFRP requires external hardware for stressing – this is felt to be impractical for bridge applications; additionally no known systems are commercially available.
4. Due to their complexity and the fact that they are untested, steel jacket repairs are not recommended; it is believed that CFRP repairs address all advantages of steel jackets while overcoming some of their drawbacks.
5. See Section 6.1.
6. See Section 6.4.1.3.
7. There are commercially available systems appropriate for bridge applications on the market in 2012 (although these may be limited).
8. Active repairs can restore some of the lost prestressing force in addition to enhancing load carrying capacity; passive repairs only affect load carrying capacity
9. See Harries et al. (2006) for a discussion of fatigue of bonded CFRP repair systems.
10. Preload may be required for the repair or simply to pre-compress associated concrete patches. U-wraps render the need to pre-compress the patch unnecessary.
11. Although data is inconclusive, U-wraps are widely believed to enhance bond performance or, at the very least to help in arresting or slowing debonding once initiated.
12. See Section 6.4.1.
13. Skills beyond those expected of a typical bridge contractor in 2012.
14. Handling equipment beyond a manlift required.
15. See Section 2.3.2

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NOTATION

A_p area of prestressing strand

b width of concrete soffit

b_f width of CFRP provided

b_{f1} width of individual CFRP strip

c depth of NSM slot

C structural capacity

C_0 capacity of undamaged girder

C_D capacity of damaged girder

C_E environmental reduction factor (ACI 440-2R 2008)

C_R capacity of repaired girder

DC, DW, LL, P and IM load effects (AASHTO *LRFD Bridge Design Specifications* (2010))

E_f modulus of elasticity of CFRP

f_c' compressive strength of concrete

f_f ultimate tensile strength of CFRP

f_{pu} ultimate tensile strength of prestressing strand

H overall depth of prestressed girder

n number of plies of CFRP

n_{max} maximum number of severed prestressing strands that can be replaced by CFRP based on its relative contribution to moment capacity

$n_{max\text{-}PT}$ equivalent number of strands whose prestress force can be replaced with PT-CFRP

$n_{spliced}$ number of prestressing strands spliced in a member

RF rating factor

RF_0 normalized rating factor (Eq. 2) of undamaged girder

RF_D normalized rating factor (Eq. 2) of damaged girder

RF_R normalized rating factor (Eq. 2) of repaired girder

t_f thickness of single CFRP ply or layer

αH depth from compression resultant to location of CFRP on soffit

βH depth from compression resultant to centroid of prestressing strands

γ_{DC}, γ_{DW}, γ_P and γ_{LL} LRFD load factors (Table 6A.4.2.2 of the *Evaluation Manual* (2011)).

ε_{fd} CFRP debonding strain

ε_{fu} ultimate tensile strain of CFRP

κ effective prestress factor in CFRP ($0 \leq \kappa \leq 1$)

APPENDIX I: Applicable national and international specifications and guides pertaining to the use and design of FRP composite repairs for strengthening, repair and rehabilitation.

United States:

National Cooperative Highway Research Program (NCHRP) - *Recommended Construction Specifications and Process Control Manual for Repair and Retrofit of Concrete Using Bonded FRP Composites* – Report 609 (2008).

National Cooperative Highway Research Program (NCHRP) - *Bonded Repair and Retrofit of Concrete Structures Using FRP Composites: Recommended Construction Specifications and Process Control Manual* – Report 514 (2008).

American Concrete Institute (ACI) Committee 440 - *ACI 440.2R-08 Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures* (2008)

Concrete Repair Manual, 3rd Ed. - published jointly by American Concrete Institute (ACI) and International Concrete Repair Institute (ICRI) (2008)

International (available in English):

Canadian Standards Association (CSA) - S806-02(R2007): *Design and Construction of Building Components with Fibre-Reinforced Polymers* (2007).

Concrete Society (Great Britain) - *Design Guidance for Strengthening Concrete Structures Using Fibre Composite Materials* - Technical Report No. 55 (2004).

International federation for Structural Concrete (fib) (European Union) - *Externally Bonded FRP Reinforcement for RC Structures* (2001).

Japan Society of Civil Engineers (JSCE) - Concrete Engineering Series 41: *Recommendations for Upgrading of Concrete Structures with Use of Continuous Fiber Sheets* (1998).

Consiglio Nazionale delle Richerche (CNR - Italy) - *Guidelines for Design, Execution and Control of Strengthening Interventions by Means of Fibre-reinforced Composites*: CNR-DT200/204 (2004).