Repair Method for Prestressed Girder Bridges

FINAL REPORT

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16. Abstract

It is common practice that aging and structurally damaged prestressed concrete bridge members are taken out of service and replaced. This, however, is not an efficient use of materials and resources since the member can often be repaired *in situ*. There are numerous repair techniques proposed by entrepreneurial and academic institutions which restore prestressed concrete girder flexural strength and save both material and economic resources. Of course, not all repair methods are applicable in every situation and thus each must be assessed based on girder geometry and the objectives of the repair scenario. This document focuses on the practical application of prestressed concrete bridge girder repair methods.

In this document, repair methods are presented for three prototype prestressed concrete highway bridge girder shapes: adjacent boxes (AB), spread boxes (SB), and AASHTO-type I-girders (IB), having four different damage levels. A total of 22 prototype repair designs are presented. Although not applicable to all structure types or all damage levels, the repair techniques covered include the use of carbon fiber reinforced polymer (CFRP) strips, CFRP fabric, near-surface mounted (NSM) CFRP, prestressed CFRP, post-tensioned CFRP, strand splicing and external steel post-tensioning. It is the authors' contention that each potential structural repair scenario should be assessed independently to determine which repair approach is best suited to the unique conditions of a specific project. Therefore, no broad classifications have been presented directly linking damage level (or a range of damage) to specific repair types. Nonetheless, it is concluded that when 25% of the strands in a girder no longer contribute to its capacity, girder replacement is a more appropriate solution. Guidance with respect to inspection and assessment of damage to prestressed concrete highway bridge girders and the selection of a repair method is presented. These methods are described through 22 detailed design examples. Based on these examples, review of existing projects and other available data, a detailed review of selection and performance criteria for prestressed concrete repair methods is provided.

Best practices based on the study objectives are presented.

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EXECUTIVE SUMMARY

Prestressed concrete bridges throughout the Commonwealth and the Nation are exhibiting signs of deterioration and distress. Recent catastrophic collapses have led to a re-evaluation of the condition of many prestressed structures resulting in new postings and in some cases emergency decommissioning of structures. Although there are many research and case studies addressing repair of prestressed bridge girders, there is little comprehensive guidance available. NCHRP Project 12-21, ultimately completed in 1985 remains the most comprehensive national study to address the evaluation and repair of prestressed bridge members. The objective of this work, therefore, is to provide an extensive state-of-the-art and state-of-practice review of both assessment and repair techniques suitable for damaged prestressed concrete bridge systems in order to prepare a "best practices" guide for the same. The emphasis is placed on structural load bearing repair techniques rather than simply aesthetic repairs.

The basis of recommendations and findings of this report is a study evaluating 22 prototype repair designs for three prototype prestressed concrete highway bridge girder shapes: adjacent boxes (AB), spread boxes (SB), and AASHTO-type I-girders (IB), having four different damage levels. Although not applicable to all structure types or all damage levels, the repair techniques covered include the use of carbon fiber reinforced polymer (CFRP) strips, CFRP fabric, near-surface mounted (NSM) CFRP, prestressed CFRP, post-tensioned CFRP, strand splicing and external steel post-tensioning.

Recommendations are organized under three general topics: bridge inspection, bridge rating and assessment, and repair design selection criteria.

Bridge Inspection Techniques
First and foremost, it is recommended that the inspection phase be separated from the assessment phase (Shanafelt and Horn 1980 and Harries 2006). The structure inspection should identify areas of damage or concern and report these quantitatively (if possible) or qualitatively (where necessary). Inspection guidance is provided to ensure that all necessary information is collected for assessment of the damaged bridge.

Bridge Rating and Assessment
Load rating and structure capacity assessments should be conducted by an engineer as a task separate from the inspection. The information provided by the inspection should be of such quality to allow the engineer to properly assess the structure's strength.

In general, the use of plane sections analysis using standard Whitney stress block factors has been shown to be adequate for assessing the capacity of damaged and repaired girders. The report describes some limitations of a plane sections approach for beams having highly eccentric loading or resistance.

It is assumed that broken strands do not contribute to the section capacity although it is proposed that allowing redevelopment of prestressed reinforcement should be considered on a case-by-case basis. A brief experimental study intended to address this effect is proposed in Section 7.2 of this report.

Based on the observed condition of the Lakeview Drive shear keys and the general inability to assess their soundness in an inspection, it is recommended that the presence of the shear keys be neglected in rating adjacent prestressed box girder bridges. Therefore, the flexural axle distribution factor is taken as $g = 0.5$ for girders less than 72" wide; and the exterior girder dead load (DC and DW) and barrier wall loading (DC) is carried entirely by the exterior girder on which it rests.

Repair Design Selection Criteria

The repair type chosen must be done so on a project-by-project basis. At this point, it is not feasible to standardize repair type selection based on damage level due to the variability between structures, the unique nature of damage to a particular girder and the original girder's design or stress requirements. A range of viable repairs for each girder type are presented without considering the specific damage level. Nonetheless, the damage level dictates which repair method can be used. Although 'percentage of strands lost' appears to be a representative indicator of girder strength, the only correlation found between percentage of lost strands to repair method has been at the level of 25% of strands lost. At this level of damage, repair (restoration of undamaged capacity) becomes impractical.

The repair method chosen is a function of the original girder's design considerations such as soffit stress, girder shape, strand spacing or layout and damage, amongst other factors. Also, the goal of the repair must be considered, i.e. if the repair must restore prestressing force (an active repair) or flexural capacity (achievable with a passive repair). Table 6.1 summarizes the potential applications and a number of selection and design considerations for each repair type. Although specific damage levels are not suggested, this table suggests the limits of applicability of each repair type.

Design of Repair and Assessment of Repair Capacity

Section 5 of this report demonstrates through 22 prototype repair examples, the design and capacity assessment of various prestressed repair methods for a range of degrees of damage. While, no broad classifications have been presented directly linking damage level (or a range of damage) to specific repair types, it is concluded that when 25% of the strands in a girder no longer contribute to its capacity, girder replacement is a more appropriate solution. Beyond this, consideration must be given to the objectives of an individual repair. In the presented examples, the objective of all repairs was to restore the undamaged capacity of the damaged girder. In some instances this was not practical. Despite some repairs failing to achieve their target capacities, the behavior of all examples was improved. This leads to three possible scenarios with respect to assessment of the repair capacity:

1. The target capacity is achieved and the repair is considered successful.
2. The target capacity is not achieved; however the beam behavior is improved sufficiently to carry required loads. The corollary of this case is that the target capacity is selected only at a level to allow the beam to perform adequately, but not necessarily achieve its original undamaged capacity. That is: the target capacity was selected only as high as is necessary to provide adequate performance.
3. The target capacity is not achieved and the beam behavior is not improved sufficiently. In this case an alternate repair method or beam replacement is required. This case permits the limit of each repair method to be assessed.

Construction specifications for CFRP systems may be based upon the recommendations of NCHRP Report 609.

While beyond the expressed scope of the present work, a brief review of methods for affecting aesthetic repairs is presented and the reader is directed to *Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products* (PCI MNL-137-06) published by the Precast/Prestressed Concrete Institute (PCI).

Recommendations for demonstration/implementation projects are presented in Section 7.

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PREAMBLE

Prestressed concrete bridges throughout the Commonwealth and the Nation are exhibiting signs of deterioration and distress. Recent catastrophic collapses have led to a re-evaluation of the condition of many prestressed structures resulting in new postings and in some cases emergency decommissioning of structures. Although there are many research and case studies addressing repair of prestressed bridge girders, there is little comprehensive guidance available. NCHRP Project 12-21, ultimately completed in 1985 remains the most comprehensive national study to address the evaluation and repair of prestressed bridge members. A 1996 Texas study and a 2004 Wisconsin study have updated the earlier guides but are limited in scope: the TXDOT study addresses only impact damage while the WIDOT study focuses primarily on corrosion mitigation techniques at girder ends in cases where strengthening or structural retrofit is largely unnecessary. No present study addresses the two primary sources of deterioration of prestressed girders: corrosion and impact – and significantly, the combination of these effects which has been demonstrated to be critical. Additionally, extant studies are necessarily out-of-date:

1. They do not address the present state of the now 25-50 year-old prestressed concrete infrastructure and the inherent deterioration associated with this aging;
2. They do not address some of the newer methods of assessing the structural capacity and, importantly, residual prestress forces; and,
3. They do not address some of the newer methods of retrofit including those using FRP materials and prestressed FRP materials.

The objective of this work, therefore, is to provide an extensive state-of-the-art and state-of-practice review of both assessment and repair techniques suitable for damaged prestressed concrete bridge systems in order to prepare a "best practices" guide for the same. The emphasis is placed on structural load bearing repair techniques rather than simply aesthetic repairs. The report is organized into sections reflecting the seven tasks of this work:

Task 1: Inventory Condition Assessment
Task 2: Review of Assessment Techniques
Task 3: Review of Repair/Rehabilitation and Retrofit Techniques
Task 4: Survey of Current State of Practice
Task 5: Representative Repair Scenarios
Task 6: Best Practices Recommendations
Task 7: Candidate Demonstration Projects

NOTATION

The following abbreviations and notation are used in this work.

Abbreviations

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
AB	Adjacent Box Beam
ACI	American Concrete Institute
CFRP	Carbon Fiber Reinforced Polymer
CFCC	Carbon Fiber Composite Cables
FRP	Fiber Reinforced Polymer
IB	I-Beam (or AASHTO Girder)
NCHRP	National Cooperative Highway Research Program

NSM	Near-surface mounted (FRP)
PCFRP	Prestressed carbon fiber reinforced polymer
SB	Spread Box Beam (or Multi Box Beam)

Notation

A_f	FRP cross sectional area
A_p	prestressed reinforcement area in the tension zone
b	width of compression face of member
C_E	environmental reduction factor
c	distance from extreme concrete compression fiber to the neutral axis
cg	strands center of gravity of strands, measured from bottom of member
d_f	effective depth of FRP flexural reinforcement
d_p	distance from the extreme concrete compression fiber to centroid of prestressed reinforcement
E_c	modulus of elasticity of concrete
E_f	tensile modulus of elasticity of FRP
E_{ps}	tensile modulus of elasticity of prestressing steel, taken as 28500 ksi
e	eccentricity of prestressing steel with respect to centroidal axis of member
f_c'	specified compressive strength of concrete
$f_c'_{DECK}$	specified compressive strength of concrete in the deck
f_{fe}	effective stress in FRP; stress level attained at section failure
f_{fu}	design ultimate tensile strength of FRP
f_{fu}^*	ultimate tensile strength of the FRP material as reported by the manufacturer
f_{ps}	stress in prestressed reinforcement at nominal strength
f_{pu}	specified tensile strength of prestressing tendons
I	moment of inertia of section
M	moment due to eccentric prestressing force in strands
M_{DECK}	moment on girder due to deck
M_{DW}	moment on girder due to wearing surface
M_{HS20}	moment on girder due to an HS20 truck
M_{HS25}	moment on girder due to an HS25 truck
M_{JB}	moment on girder due to Jersey barrier
M_{LANE}	moment on girder due to AASHTO (2007) lane load
M_n	nominal flexural strength of girder
M_{nf}	contribution of FRP to nominal flexural strength of girder
M_{np}	contribution of prestressing steel to nominal flexural strength of girder
M_{SW}	moment on girder due to its self-weight
M_{TAN}	moment on girder due to AASHTO (2007) tandem load
M_u	design ultimate flexural strength of girder
n	number of plies of FRP reinforcement
P_e	effective force in prestressing reinforcement (after all losses)
r	radius of gyration of a section
S	section modulus
t_f	nominal thickness of one ply of FRP reinforcement
y_b	distance from extreme bottom fiber to the section centroid
y_t	distance from top fiber to the section centroid
α	empirical constant to determine an equivalent rectangular stress distribution in concrete
β_1	ratio of depth of equivalent rectangular stress block to depth of neutral axis

ε_{bi} strain level in concrete substrate at time of FRP installation (tension is positive)
ε_c strain level in concrete
ε_c' maximum strain of unconfined concrete corresponding to f'c; may be taken as 0.002
ε_{cu} ultimate axial strain of unconfined concrete
ε_{fd} debonding strain of externally bonded FRP reinforcement
ε_{fd}* debonding strain of externally bonded PT FRP reinforcement
ε_{fe} effective strain level in FRP reinforcement attained at failure
ε_{fu} design rupture strain of FRP reinforcement
ε_{fu}* ultimate rupture strain of FRP reinforcement
ε_{pe} effective strain in prestressing steel after losses
ε_{pi} initial strain level in prestressed steel reinforcement
ε_{pnet} net strain in flexural prestressing steel at limit state after prestress force is discounted (i.e.: excluding strains due to effective prestress force after losses)
ε_{ps} strain in prestre'ssed reinforcement at nominal strength
ε_{pt} strain induced in FRP reinforcement by PT
ψ_f FRP strength reduction factor

This report provides values in US units (inch-pound) throughout. The following "hard" conversion factors have been used:

1 inch = 25.4 mm
1 kip = 4.448 kN
1 ksi = 6.895 MPa

Reinforcing bar sizes are reported using the designation given in the appropriate reference. A bar designated using a "#" sign (e.g.: #4) refers to the standard inch-pound designation used in the United States where the number refers to the bar diameter in eighths of an inch.

DISCLAIMER
This document presents engineering design examples; use of the results and or reliance on the material presented is the sole responsibility of the reader. The contents of this document are not intended to be a standard of any kind and are not intended for use as a reference in specifications, contracts, regulations, statutes, or any other legal document. The opinions and interpretations expressed are those of the author and other duly referenced sources. The designs presented have not been implemented nor have they been sealed by a professional engineer.

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TASK 1: INVENTORY CONDITION ASSESSMENT

A review of all prestressed concrete bridge structures in Pennsylvania was conducted. All bridges having a 'structure type' coded 4xxxx (i.e.: prestressed concrete) in the PONTIS database were included. Data was considered on a statewide basis (including District 11) and for District 11 (Allegheny, Beaver and Lawrence counties) only. The intent of this exercise was to establish a snapshot of the condition of the prestressed concrete bridge inventory in Pennsylvania and to ensure that the bridges considered for further study (from District 11) were representative of the statewide distribution.

1.1 Bridge Inventory Fields Reviewed

BMS Number	Min. Vert. Clearance Right	Deck Rating	Deck Width
		Super Rating	
Structure Type	NHS IND	Sub Rating	Suff. Rate
Feature Carried	Insp Date	Culvert Rating	C/P/A
Feature Under Bridge	Insp Freq	SD/FO	Hwy Sys Type
On/Under Indicator	Year Built	ADT	Culvert Length
Min. Vert. Clearance Left	Year Recon	Length	Deck Area

Table 1.1 provides a summary of the data obtained based on bridge type considering statewide and District 11 data. For this exercise, only structures rated as 'structural deficient' (SD) are considered. Additionally, the data is divided into those bridges rated deficient for 'any' (deck, superstructure, substructure) reason and for only superstructure deficiency ('super'); the latter is the focus of the present study. In reading Table 1.1, the percentages reported in the 'No.' columns are determined based on the total number of prestressed bridges reported; thus statewide, 33% of prestressed bridges are 'simple composite multi-box beams' (1921/5874 = 0.33). The percentages reported in the 'SD' columns are based on the total number of bridges of a particular type; thus statewide, 11% of the 'simple composite multi-box beams' are structurally deficient (214/1921 = 0.11). The following observations are made based on this data:

- Statewide, the inventory of prestressed bridges has proportionally fewer deficient structures (15.1%) than the total inventory (21.4%). This should be expected since prestressed concrete is a relatively durable material and the average age of the prestressed inventory is younger than the inventory as a whole.
- District 11 has a greater proportion of prestressed bridges (37.7%) than the statewide inventory (23.3%).
- District 11 reports a greater proportion of deficient structures (28.4%) than the statewide inventory. Additionally the proportion of prestressed bridges reported as being deficient in District 11 (28.0%) is comparable to the total inventory deficient in this district (28.4%). However, the majority of deficient structures in District 11 are not rated as deficient based on their superstructure condition and District 11 has essentially the same proportion of deficient prestressed superstructures as the statewide inventory (7.8% in each case).
- Four bridge types dominate the prestressed inventory: simple, noncomposite adjacent box beams (14% of prestressed inventory statewide and 10% in District 11); simple composite I-beams

(22%/25%); simple composite multi-box beams[1] (33%/26%); and simple composite adjacent box beams (19%/14%).

- Considering only prestressed bridges rated deficient based on their superstructure rating, noncomposite adjacent box beams represent the majority of such bridges (40% of such bridges are deficient statewide representing 71% of the deficient prestressed structures in the state). Composite I-beam, adjacent box beam and multi-box beams also represent large numbers of such deficient bridges. The trends and the dominance of these four bridge types are similar when considering District 11 only.

Based this review, 28 bridges from District 11 were selected for an in-depth review of their inspection reports in order to assess the nature of damage resulting in a 'structural deficient' superstructure rating. As indicated in Table 1.1, five bridge types[1], reflective of the District 11 inventory were selected. Initially, 22 bridges (Bridges A – H in Table 1.2) were selected based on a) having a superstructure rating less than 4; and b) having low reported clearance over a roadway. The latter criterion was selected to ensure some vehicle impact damage would be present in the sample. Five additional bridges having known vehicle impacts were added (Bridges J – P). Finally, the collapsed Lake View Drive bridge (Harries 2006) from District 12 was added. Table 1.2 summarizes the 29 bridges selected for further study. The bridges have been assigned an alphanumeric identification as shown in Table 1.2 which will be adopted for clarity in further reporting and to obscure the identity of in-service bridges in publications beyond this report.

1.2 Sources of Damage to Prestressed Bridge Girders

Observed ***sources*** of damage to prestressed concrete girders are classified as indicated in Table 1.3. Vehicle impact damage (Source I) was the basis for bridge selection and is thus disproportionately represented in the sample. As of July 16, 2008, only 18 bridges in District 11 were listed as having undergone significant damage from vehicle impact; 7 of these were prestressed concrete structures. Impact damage (Figures 1.1 to 1.5) ranges from significant loss of section and reinforcing (Figure 1.1), which was not observed in the bridges investigated, to minor 'scrape' marks on the bridge soffit (Figure 1.2). Impact may result in spalling, typically resulting in exposed (although rarely damaged) strands (Figures 1.3 and 1.4). Feldman et al. (1996) identified a commonly occurring damage pattern associated with side impact. The impact causes a torsion-induced shear cracking pattern in the exterior (or fascia) girder as shown in Figure 1.5. This was observed in Bridge P, reviewed for this study (Figure 1.5).

The most common source of damage observed results from 'environmental distress' and simple aging of the structure coupled with limited or inadequate maintenance (Source II). Chloride intrusion resulting from the use of road salt is the most significant environmental stressor. Chloride-laden water from the bridge surface may affect the bridge deck, sides of the bridge and soffit region where no 'drip strips' are present (Figure 1.6). Additionally, chlorides may be introduced into regions assumed to be 'protected' as a result of leaking expansion joints and drain systems (Figure 1.7). Deterioration of shear keys in adjacent box girders (observed in the Lake View Drive bridge (Harries 2006) and anecdotally throughout southwestern Pennsylvania[2]) results in chloride laden water accessing all webs and most of the soffit

[1] There is some confusion in the inventory. 'Simple noncomposite multi-box beams' are reported although there is not believed to be such a structure type. It is believed that this classification represents a mis-classification either 'simple composite multi-box beams' or 'simple noncomposite adjacent box beams'.

[2] Many noncomposite adjacent box girders display icicles between their beams during winter. These icicles are often 'stained' indicating some degree of active corrosion.

(Figure 1.6). Spray from trucks travelling beneath the bridge may introduce additional chloride-laden water to the underside of the bridge superstructure. Although not an issue in the present study, bridges located near an ocean environment are also subject to enhanced chloride attack. Related to the presence of water (whether chloride-laden or not), is the potential for damage associated with freezing and thawing cycles. Such freeze/thaw damage in prestressed structures typically requires other damage to be present (allowing water ingress) before initiating.

Improper retrofit or repair practices can initiate damage (Source III). For example, a concrete patch having a lower chloride content than the adjacent concrete can result in the formation of a localized corrosion cell at the patch interface resulting in accelerated corrosion in this region even without further chloride load (as the chloride ions migrate from the older concrete into the patch). This source of damage is most commonly observed on patched decks. Another damage source (IV) associated with bridge retrofit was observed where a barrier rail system was replaced and the original bolted attachment locations not patched. This led to local spalling as shown in Figure 1.8. Additionally, the possibility that the new rail mounting (Figure 1.8a) is drilled through a strand or may cause future spalling cannot be discounted.

Inadequate maintenance practices may not be a primary source of damage; however they will exacerbate existing damage (Source V). Clogged drain systems, exposed strands, concrete that remains un-patched and clogged weep holes are all maintenance issues that must be corrected before further damage results. For example, weep holes in the adjacent box girders of the Lake View Drive Bridge (Harries 2006) were clearly clogged as evidenced by significant water residing in the beam voids (collapsed void forms can be seen in Figure 1.9). This internal water may affect chloride attack of the girder soffit from the top-down (not observed in the Lake View Drive bridge) and adds an unaccounted-for dead load to the girder.

Construction error (Source VI) may result in bridge damage if uncorrected. Minor errors may exacerbate degradation from other sources. For example, Figure 1.9 shows that some strands in the Lake View Drive Bridge had only one half of their prescribed 1.5 inch concrete cover. Such misplacement results in less protection to the steel from chloride intrusion and is likely to exacerbate spalling.

Bridges may be damaged by overload (Source VII) or extreme events (Source VIII). Such loads may be from overloaded or oversized vehicles or from natural causes including seismic effects (Figure 1.10a) or floods. In general, damage from flood-borne debris will be similar to that caused by vehicle impact but may be located anywhere in the bridge depth. No such damage was observed in the present study. Bridges may also be damaged by fire (Figure 1.10b). Due to the nature of such damage, bridges affected by fire should be assessed on a case-by-case basis. Fire damage is beyond the scope of the present work.

1.3 Types of Damage to Prestressed Bridge Girders

Observed ***types*** of damage to prestressed concrete girders are classified as indicated in Table 1.4. This classification may be interpreted as a damage continuum. Left uncorrected, less significant damage types (Types i and ii) will progress to becoming more significant (Types iii to v) as corrosion becomes manifest. Eventually corrosion will lead to section loss of the strand (Types vi and vii) and resulting loss of prestress and member capacity. Figure 1.11 schematically illustrates this continuum of corrosion damage. In general, the progression of corrosion-related damage tends to be exponential in time. Repairing such ***types*** of damage must be accompanied my mitigating the ***source*** of the damage where possible.

Mechanical damage resulting in strand rupture may also result from significant impact events (Type viii) or other overloads (Types ix to xi), although the latter are rare and not generally observed in the present study. It should be noted that the load tests carried out on girders recovered from the Lake View Drive Bridge (Harries 2006) resulted in examples of both shear (Type ix) and flexural (Type x) damage as shown in Figures 1.12 and 1.13, respectively. Longitudinal cracking (Type xi) may result from impact (Fig. 1.5) or from corrosion of reinforcement prior to spalling. The latter will generally be accompanied by staining. Longitudinal cracking was apparent prior to the May 2000 collapse of a prestressed double-tee pedestrian bridge at Lowes Motor Speedway in Charlotte NC[3].

[3] The author of this report investigated this collapse although cannot provide a specific citation or images due to litigation issues.

Table 1.1 Summary of statewide and District 11 prestressed bridge inventory.

	Structure Type Code	Statewide			District 11[1]			bridges considered for further study[5]	
		No.	SD (rating < 4)		No.	SD (rating < 4)			
			Any[2]	Super		Any[2]	Super	review	design
all bridges[3]	xxxxx	25203	5385 (21.4%)	3465 (13.7%)	1781	505 (28.4%)	318 (17.9%)		
all prestressed[4]	4xxxx	5874 (23.3%)	887 (15.1%)	456 (7.8%)	671 (37.7%)	188 (28.0%)	52 (7.8%)		
simple, noncomposite slab	4x101	42	3 (7%)	2 (5%)	0	0	0		
simple, noncomposite hollow slab	4x102	4	2 (50%)	0	4	2 (50%)	0		
simple, noncomposite I beam	4x104	56	16 (29%)	1 (2%)	29	15 (52%)	0	2	x
simple, noncomposite multi-box beam[8]	4x106	84	20 (24%)	11 (13%)	41	16 (39%)	9 (22%)	9[6]	x
simple, noncomposite adjacent box beam	4x107	821 (14%)	350 (43%)	326 (40%)	69 (10%)	19 (28%)	14 (20%)	6[7]	x
simple, composite slab	4x201	55	1 (2%)	0	6	0	0		
simple, composite I beam	4x204	1275 (22%)	173 (14%)	29 (2%)	167 (25%)	59 (35%)	9 (5%)	4	
simple, composite multi-box beam	4x206	1921 (33%)	214 (11%)	55 (3%)	177 (26%)	53 (30%)	12 (7%)	5	
simple, composite adjacent box beam	4x207	1110 (19%)	95 (9%)	29 (3%)	95 (14%)	17 (18%)	8 (8%)	3	
simple, composite other	4x299	3	1 (33%)	0	1	0	0		
continuous, noncomposite I beam	4x304	5	0	0	3	0	0		
continuous, noncomposite multi-box beam[8]	4x306	1	0	0	0	0	0		
continuous, noncomposite adjacent box beam	4x307	1	0	0	0	0	0		
continuous, composite I beam	4x404	210	7 (3%)	0	50	7 (14%)	0		
continuous, composite multi-box beam	4x406	197	0	0	20	0	0		
continuous, composite adjacent box beam	4x407	65	1 (2%)	0	9	0	0		
other I beam	4x504/804	6	1 (17%)	0	0	0	0		
other multi-box beam	4x806	5	0	0	0	0	0		
other adjacent box beam	4x807/907	10	3 (30%)	3 (30%)	0	0	0		
other	4xxxx	2	0	0	0	0	0		

[1]Allegheny, Beaver and Lawrence Counties
[2]Deck, Superstructure and Substructure only (culverts not considered)
[3]data from September 10, 2007
[4]prestressed data from: statewide: February 12, 2008; District 11: December 26, 2007
[5]only bridges from District 11 were considered for further study
[6]more 4x106 bridges were selected for review as many had vertical clearance issues
[7]includes Lake View Drive Bridge.
[8]there is not believed to be such a structure as a *noncomposite multi box beam*. It is believed that this classification represents a mis-classification either *simple composite multi-box beams* (4x406) or *simple noncomposite adjacent box beams* (4x107).

	Table 1.2 Bridges Selected for further investigation of inspection records.											
ID	BMS Number	Structure Type		Feature Carried	Feature Intersected	Min Vert. Clear (ft)	Year		Rating			Suff. Rate
							Built	Recon.	Deck	Super	Sub	
A	02-0376-0130-1385	S-NC-multi box beam	42106[1]	Parkway East	Old W.Penn H	17.25	1962	1976	5	3	4	27.10
A	02-0376-0130-1385	S-NC-multi box beam	42106[1]	Parkway East	Parkway East	99.90	1962	1976	5	3	4	27.10
A	02-0376-0130-1385	S-NC-multi box beam	42106[1]	Parkway East	Parkway East	99.90	1962	1976	5	3	4	27.10
A	02-0376-0130-1385	S-NC-multi box beam	42106[1]	Parkway East	Leak Run	53.00	1962	1976	5	3	4	27.10
B	02-0885-0230-0000	S-NC-multi box beam	42106[1]	Mifflin Rd	Route 885	99.90	1967	-	4	4	4	47.30
B	02-0885-0230-0000	S-NC-multi box beam	42106[1]	Mifflin Rd	SR 8059	14.58	1967	-	4	4	4	47.30
C	02-0885-0221-1126	S-NC-multi box beam	42106[1]	Mifflin Rd	Mifflin Rd	99.90	1963	-	5	4	4	49.00
C	02-0885-0221-1126	S-NC-multi box beam	42106[1]	Mifflin Rd	Glass Run Rd	14.42	1963	-	5	4	4	49.00
C	02-0885-0221-1126	S-NC-multi box beam	42106[1]	Mifflin Rd	Glass Run Rd	14.42	1963	-	5	4	4	49.00
D	02-2046-0040-1678	S-NC-adjacent box beam	42107	Streets Run Rd	Streets Run Rd.	99.90	1957	-	4	3	5	41.30
D	02-2046-0040-1678	S-NC-adjacent box beam	42107	Streets Run Rd	Streets Run	10.00	1957	-	4	3	5	41.30
E	02-2046-0060-0780	S-NC-adjacent box beam	42107	Streets Run Rd	Strts.Rn.Rd.	99.90	1901	1957	5	4	5	22.70
E	02-2046-0060-0780	S-NC-adjacent box beam	42107	Streets Run Rd	Streets Run	8.00	1901	1957	5	4	5	22.70
F	04-4020-0030-0657	S-C-I beam	42204	Dutch Ridge Rd	LR 1023	32.00	1969	-	3	4	4	63.10
F	04-4020-0030-0657	S-C-I beam	42204	Dutch Ridge Rd	LR 1023	32.00	1969	-	3	4	4	63.10
F	04-4020-0030-0657	S-C-I beam	42204	Dutch Ridge Rd	Dutch Ridge	99.90	1969	-	3	4	4	63.10
G	04-4035-0040-0525	S-C-multi box beam	42206	Sebring Rd	LR 1023	14.75	1973	-	3	4	4	56.50
G	04-4035-0040-0525	S-C-multi box beam	42206	Sebring Rd	LR 1023	14.75	1973	-	3	4	4	56.50
G	04-4035-0040-0525	S-C-multi box beam	42206	Sebring Rd	Sebring Rd	99.90	1973	-	3	4	4	56.50
G	04-4035-0040-0525	S-C-multi box beam	42206	Sebring Rd	Sebring Rd	99.90	1973	-	3	4	4	56.50
H	02-3051-0050-0295	S-C-adjacent box beam	42207	Half Crown Rd	Rt.22	15.58	1966	-	3	4	3	33.00
H	02-3051-0050-0295	S-C-adjacent box beam	42207	Half Crown Rd	LR.396	99.90	1966	-	3	4	3	33.00
H	02-3051-0050-0295	S-C-adjacent box beam	42207	Half Crown Rd	Rt.22	15.58	1966	-	3	4	3	33.00
J	02-0279-0144-0000	S-C-multi box beam	42206		Camp Horne Rd	15.00	1988	-	-	5	-	80.00
K	02-3084-0020-1033	S-NC I beam	42104		LR 1023 – TR 60	14.42	1970	-	-	5	-	63.60
M	02-4022-0020-0000	S-NC I beam	42104		Interstate 79	15.92	1971	-	-	5	-	43.60
N	37-4012-0010-0698	S-C-I beam	42204		LR 1023 – PA 60	14.42	1970	-	-	5	-	48.80
P	04-0060-0160-1497	S-NC-adjacent box beam	42107									
LV	Lake View Drive[2]	S-NC-adjacent box beam	42107	Lake View Drive	I-70	14.50	1961	-	-	-	-	-

[1]there is not believed to be such a structure as a *noncomposite multi box beam*. It is believed that this classification represents a mis-classification either *simple composite multi-box beams* (42406) or *simple noncomposite adjacent box beams* (42107).

[2] the Lake View Drive Bridge collapsed December 27, 2005

Table 1.3 Sources of observed damage.

Damage Source	Description	Representative Photograph(s)	Bridges where observed
I	Impact by over height vehicle	Figs. 1.1 to 1.5	A, C, J-P & LV
II	Environmental Distress/Aging including freeze-thaw and water-induced	Figs. 1.6 and 1.7	A, E, F, G, H, N & LV
III	Construction error or poor practice associated with previous repair	-	H & LV
IV	Construction error associated with appurtenance mounting	Fig. 1.8	C & E
V	Poor maintenance practice	Figs 1.7 and 1.8	A, C, E, F, H & LV
VI	Construction error	Fig. 1.9	LV
VII	Load-related damage (other than impact), including effects of natural disasters	Figs. 1.12 and 1.13	E
VIII	Extreme events such as natural disaster and fire	Fig. 1.10	none

Table 1.4 Types of observed damage.

Damage Type	Observed Damage	Representative Photograph(s)	Bridges where observed	Damage Source
i	Concrete spalling	Fig 1.11	A, C, D, E, F, G & LV	all
ii	Exposed prestressing strands		A, C, D, E, F, G, K, N & LV	all but VI
iii	Corroded prestressing strand without pitting		A, E, J, N & LV	all but VI
iv	Corroded prestressing strand with light pitting		A, LV	all but VI
v	Corroded prestressing strand with heavy pitting		A, LV	all but VI
vi	Partial loss of strand area due to corrosion (rupture of individual wires)		A, LV	all but VI
vii	Complete loss of strand area due to corrosion		A, LV	all but VI
viii	Strand rupture associated with load or impact	Figs 1.3 – 1.4	K, N &LV	I, IV, VII & VIII
ix	Shear cracking of girder	Fig. 1.12	C, G & LV	I, VI, VII &VIII
x	Flexural cracking of girder	Fig. 1.13	none	VI, VII & VIII
xi	Longitudinal cracking of girder	Figs 1.3(c) and 1.5	J, N & P	I, II, VII,& VIII

Figure 1.1 Loss of section of AASHTO girder due to vehicle impact (Harries; not taken in PA).

Figure 1.2 'Scraping' due to minor vehicle impact (Lake View Drive Bridge prior to collapse; PennDOT and Harries 2006).

(a) damage to girder soffit.

(b) close up view of (a) showing severed strands.

(c) longitudinal cracking resulting from impact.

Figure 1.3 Impact damage to I-beam (PennDOT).

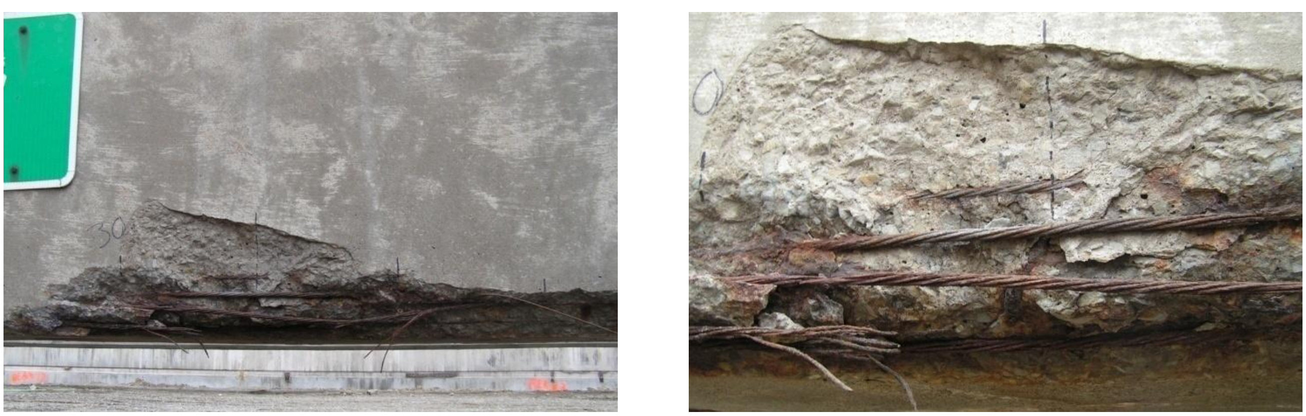
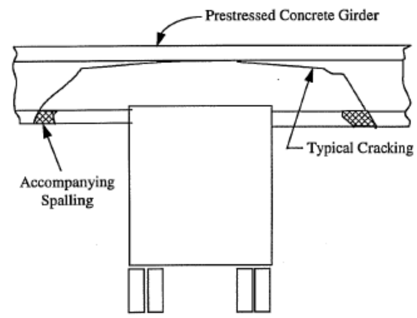


Figure 1.4 Exposed and ruptured strand due to vehicle impact (Lake View Drive Bridge; Harries 2006).

(a) following vehicle impact (PennDOT).

(b) typical impact damage pattern (PennDOT).



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

Figure 1.5 Vehicle impact due to collision.

(a) water coming down exterior face of adjacent box girder (Harries 2006).

(b) water leaking between adjacent box girders (PennDOT).

Figure 1.6 Evidence of water on soffits of adjacent box girders.

(a) water pooling due to clogged deck drain (PennDOT).

(b) damaged drain system resulting in water affecting superstructure (PennDOT).

Figure 1.7 Water may come from unanticipated sources.

(a) spalling at original attachment and possible future damage at sight of new attachment

(b) unpatched holes at sight of original attachment result in exposed strands.

Figure 1.8 Damage to strands caused by relocating barrier supports (PennDOT).

¾" center of strand to soffit

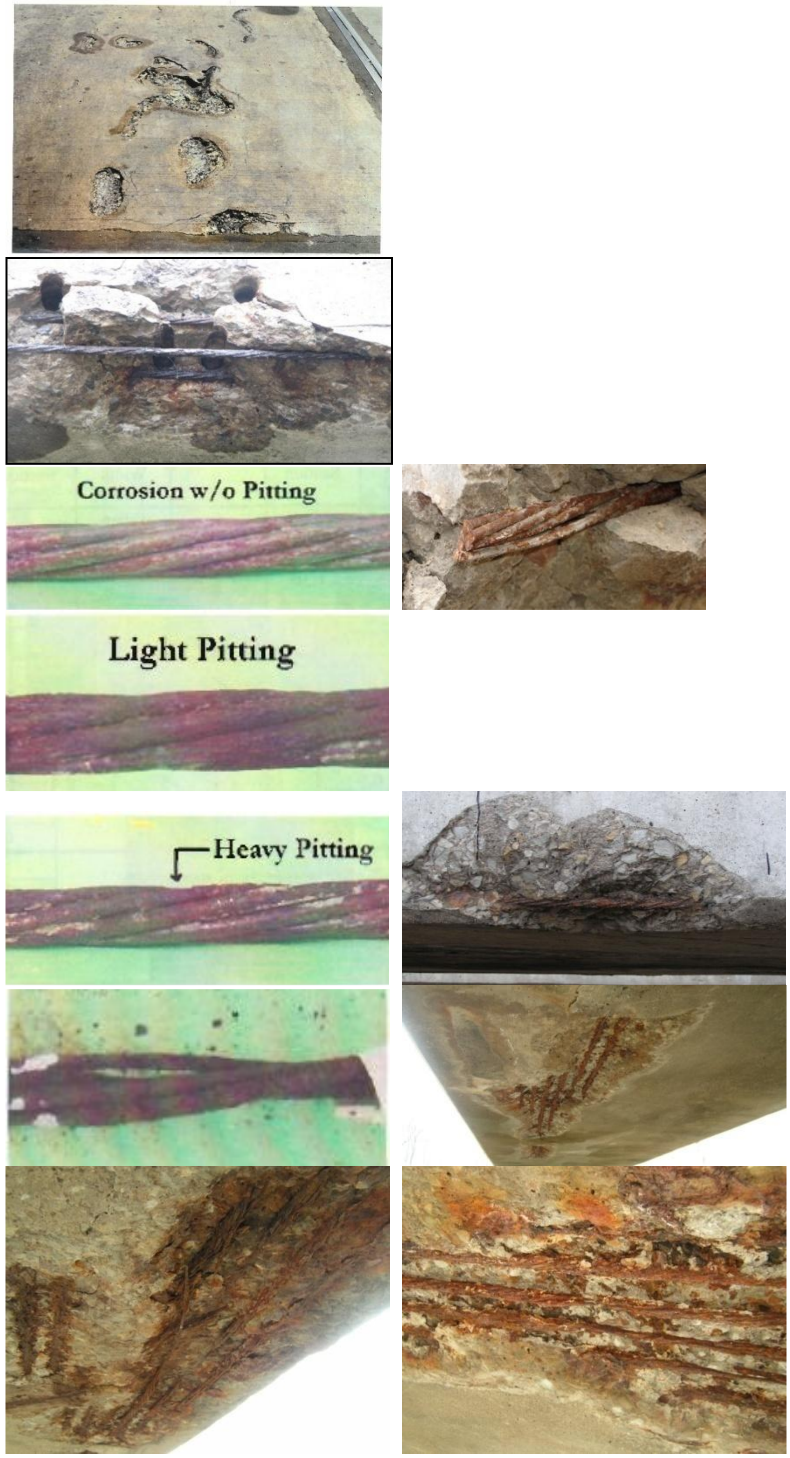
inconsistent spacing

Figure 1.9 Girder with insufficient cover and inconsistent strand spacing (Lake View Drive Bridge; Harries 2006).

(a) Earthquake (FEMA).

(b) fire (SIKA Corporation).

Figure 1.10 Damage due to extreme events – beyond the scope of the present report.



i: Concrete spalling

ii: Exposed strands without corrosion (Fig 1.6b)

iii: Corroded strand without pitting (strand intentionally cut)

iv: Corroded strand with light pitting

v: Corroded strand with heavy pitting

vi: Partial loss of strand area

vii: Complete loss of strand area

Figure 1.11 Continuum of corrosion damage (Naito et al 2006; Harries 2006)

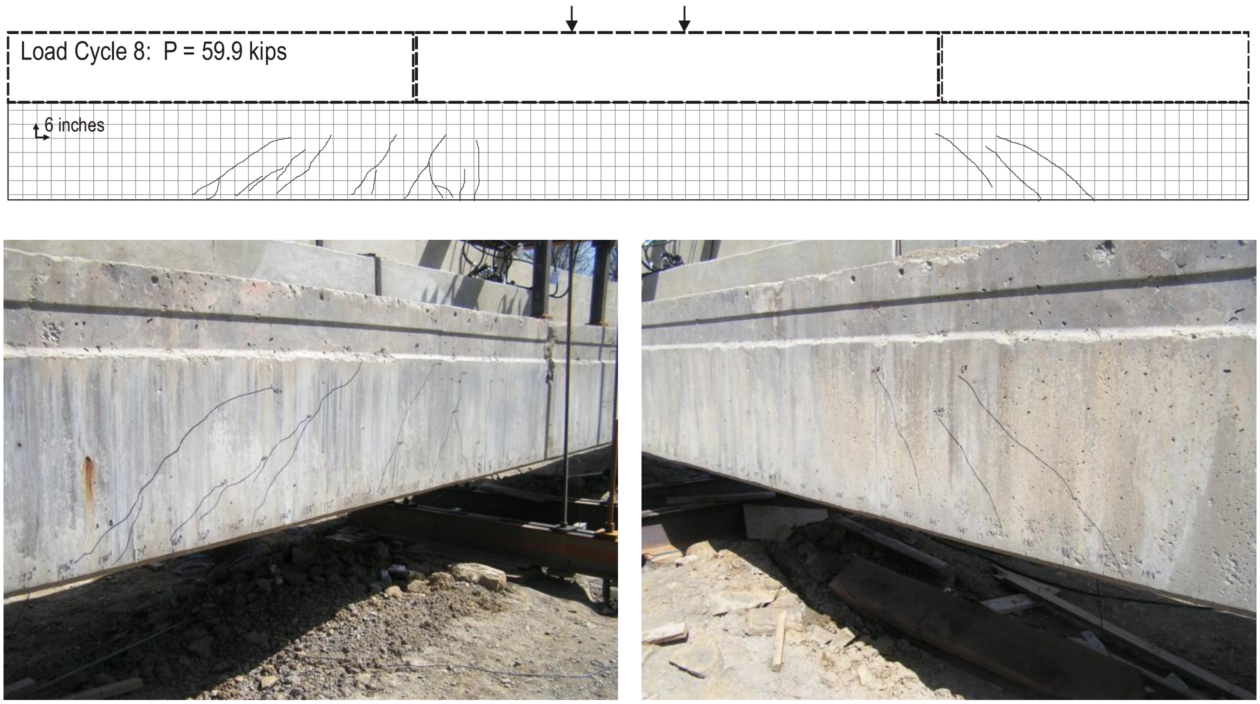


Figure 1.12 Representative shear distress (Lake View Drive EXTERIOR test girder; Harries 2006).

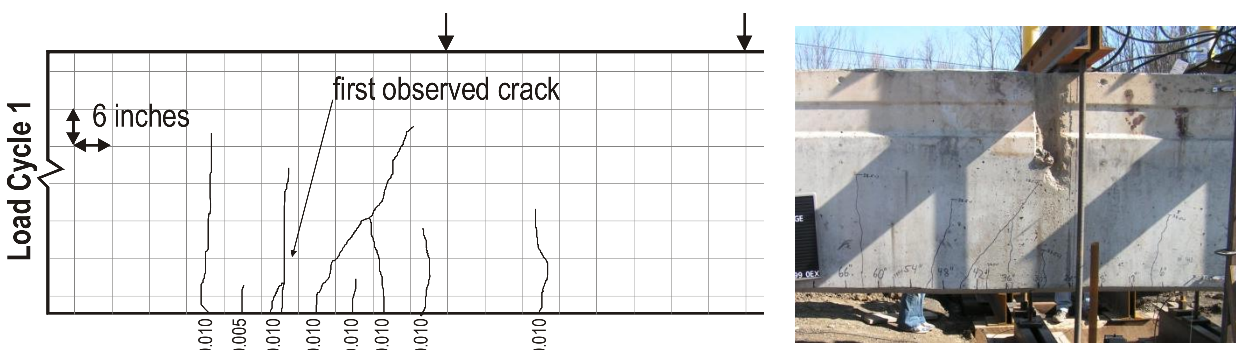


Figure 1.13 Representative flexural distress (Lake View Drive INTERIOR test girder; Harries 2006).

TASK 2: REVIEW OF ASSESSMENT TECHNIQUES

A review of non-destructive testing and evaluation (NDT/NDE) techniques for assessing prestressed concrete elements was undertaken. The following table summarizes this review. A brief summary of the appropriateness of each method is included in the section header.

Reference	Sponsoring Agency	Structure Type	Findings/Discussion/Summary
VISUAL INSPECTION In general, visual inspection is the only practical triage tool; sites requiring further non-destructive or destructive evaluation are identified. Nonetheless, a skilled inspector, familiar with the structure she is inspecting will still provide a remarkably accurate assessment of the condition of the structure although is unlikely to be able to accurately quantify many damage types.			
Shanafelt and Horn 1980 and 1985	NCHRP 12-21	Girders	This report categorizes physical damage into 3 separate categories: minor, moderate, severe. Detailed assessment of damage such as damage caused by corrosion of strands or the remaining effective prestress was not mentioned. The initial damage inspection is conducted visually with the help of some tools such as a chipping hammer, magnifying glass, ultrasonic test equipment, concrete coring equipment, etc. *The findings of these reports are discussed further in Section 3.1*
Mehta and Monteiro 2006			A simple method of assessing the condition of concrete is to tap the surface with a hammer and listen to the resulting tone. A high frequency pitch indicates a sound concrete whereas a lower frequency pitch indicates the presence of flaws. This simple method combined with visual inspection of concrete surface for cracks extending over the reinforcement length may provide preliminary information about the structure. However, it is dependent on the skill level of the operator and does not provide information about the extent of damage.
Walther and Hillemeier 2008	German Dept. for Urban Development	Visible Tendons	The disadvantage of visual inspection is its local nature. Only a part of the tendons can be inspected which may not represent the whole tendon. Furthermore, the chipped section of concrete must be repaired afterwards, which might provide a focal point for future corrosion. The 'screw-driver-test' tests the state of the tendon by trying to wedge a flat-head screw driver between its wires. If the wires are arranged in a bundle, only a very limited number of wires can be visually inspected.
RADIOGRAPHY While yielding tremendous results, this approach is currently impractical for field application. Application of radiographic techniques has also been reported by Petrou et al. (2000) in conjunction with freshly placed concrete.			
Ali and Maddocks 2003	GHD Company	Should be accessible on both sides	Radiographic examination involves the use of a powerful radiation source to produce x-rays. Access to both sides of the structure is required in this method. Visualization may be achieved by either radiographs or real-time imaging. For radiographs, radiographic film is exposed for up to 30 minutes. The practical limit of concrete thickness for this method is 600 mm. For real-time imaging, French Scorpion System is available.

Reference	Sponsoring Agency	Structure Type	Findings/Discussion/Summary
COMPUTED TOMOGRAPHY This approach is currently impractical for field application.			
Ali and Maddocks 2003	GHD Company	Should be accessible on both sides	Measures the attenuation of an incident beam that travels in a straight path in an object. The method can only be used when there is significant loss of section due to corrosion. Surface temperature variations and other climatic conditions affect results. The resolution limit of 1 mm and also the contrast resolution should be improved in order to use it for corrosion determination. Data collection and processing is slow.
Mehta and Monteiro 2006			Computed tomography using x-rays, electrical impedance tomography, and backscattering microwave imaging are methods applicable to concrete. X-ray computed tomography shows promise in detection of cracks and in locating rebar in moderate size structures. However, it requires elaborate work and is costly for setup and operation. Electrical impedance tomography is fast, inexpensive, easy to use, and has the potential for locating rebar, water-filled fractures, and possibility of gaining information about corrosion around reinforcement. Microwave imaging can be used to locate rebar within 6 cm of the surface. Although it is limited to shallow sections, it is fast, simple and relatively inexpensive.
SURFACE PENETRATING RADAR Not well suited to assessing state of deterioration/damage although appropriate for identifying *in situ* conditions if these are not known.			
Ali and Maddocks 2003	GHD Company		Works on the principle of reflection of high frequency electromagnetic pulses from interfaces between materials with different dielectric constants such as steel, concrete and voids. The transducer is passed over the section to locate the position and depth of tendons. However, this technique can only give relative results and is not suitable for detecting corrosion. It should be used together with destructive methods for an estimation of tendon loss.
IMPACT-ECHO Not currently practical for field implementation although deterioration of concrete beam subject to fatigue loads has been successfully demonstrated (Zein et al. 2008)			
Ali and Maddocks 2003	GHD Company	Requires a smooth surface	A stress pulse is introduced in the structure by a small impact. A transducer nearby collects the reflected stress wave. The time of travel is calculated from the reflected stress wave and the corresponding frequencies denote the locations of the reinforcement or voids. The main difference from radar is the use of low frequency waves (up to 60 kHz) which solves some problems related to non-homogeneity of concrete. Although the method is relatively successful in locating voids and reinforcement, it is not suitable for the use of assessing corrosion.
Zein, Gassman and Harries (2008)	NSF		Demonstrated impact-echo method to determine deterioration of decommissioned 40-year-old Interstate girders subject to fatigue loads in a laboratory environment. Apparent deterioration of concrete modulus as function of load correlated well with physical measures of accumulated damage (strain and displacement).

Reference	Sponsoring Agency	Structure Type	Findings/Discussion/Summary
ACOUSTIC EMISSION Acoustic emission is a viable method of continuous structural health monitoring. Alternate approaches using known applied loads (trucks) have been demonstrated to be viable methods of inspection (as different from monitoring). The methods are very promising and viable for 'problem structures', although some technical hurdles remain before wide-spread deployment is practical.			
Ali and Maddocks 2003	GHD Company	Several sensors are attached on a structure	The method is based on the principle that waves are emitted when damage occurs to a strand. Continuous monitoring is carried out with sensors. Monitoring can be used to determine new cracks but will not provide information on previously existing damage.
Mehta and Monteiro 2006			AE is a noninvasive, nondestructive method that analyzes noises that are created when materials deform or fracture. Events occurring inside the structure create waves that are collected at the surface by receivers.
Ramadan et al. 2007		Only tested on cold-drawn steel specimens in the lab.	The method is applicable for real-time health monitoring of a bridge or a girder. No information available for past fractures or the current degree of damage until failure.
Gostautus and Harries 2008	PennDOT	prestressed adjacent box girder	Acoustic emission was very successfully used to quantify and locate damage to two prestressed box girders tested to failure (Harries 2006 and Figs 1.12 and 1.13).
MAGNETIC FIELD DISTURBANCE This approach is currently impractical for field application, although improvements in resolution are expected to improve its viability.			
Ali and Maddocks 2003	GHD Company	Location of reinforcement must be known	The method consists of applying a constant magnetic field to the member and scanning all assessable surfaces for any abrupt change in the field which might be due to a flaw in the strands. This method is able to detect corrosion and strand failures but is limited in its resolution and accuracy. Each type of defect: pitting, notches, loss of section, etc. has its own unique signature which enables the user to distinguish between them. However, stirrups and other metallic objects embedded in the structure may hide the actual metal loss or defects and lead to errors. The detectable defect size is 5% loss of section when there are no stirrups. When stirrups are present, there is a loss in resolution, and the detectable defect size increases to 40% loss of section when the stirrups are spaced at 400 mm. With further decrease in the spacing of stirrups, the method loses its capacity to give accurate results. A high level expertise is required to interpret the results. Furthermore, knowledge of the structure such as locations of reinforcement is required to be able to interpret results.
REMNANT MAGNETISM Commercially viable systems available primarily aimed at detection of flaws/damage in prestressed slabs. Available systems could be readily adapted to high-speed applications on bridge soffits. Very promising in near-term.			
Ali and Maddocks 2003	GHD Company		The prestressing steel strands are magnetized up to saturation to remove their magnetic history. This process is performed by an electromagnet along the direction of the tendon. Fractures and breaks in the prestressing tendons are detectable but the size of the defect or loss of section is not.

Reference	Sponsoring Agency	Structure Type	Findings/Discussion/Summary
Scheel and Hillemeier 2003	Berlin Building Authority		The method is useful to get information about the location of prestressing steel fractures and the degree of damage to a strand. Limitations mainly depend on the density of the reinforcement and the minimum degree of damage that is sought. The method can be applied on the vertical face or the bottom face of a member. The magnetic properties of the steel change with different levels of prestressing. Fracture can be detected even if it is screened by other wires or if the resulting gap in the steel is relatively small. It is planned to increase the measurement speed by magnetizing the tendons by using a large mobile magnet and then scanning the magnetic flux of the entire surface.
Walther and Hillemeier 2008	German Dept. for Urban Development	Prestressed decks	In this non-destructive method, the tendons are magnetized with an electromagnet placed on the surface of concrete and then the magnetic leakage fields are monitored for any irregularities which might be caused by fractures in wires. Individual fractured wires can be detected even if they are inside a bundle of several intact wires. When testing tendons from the upper surface, it is possible to speed up the process with the use of a large mobile magnet to magnetize several tendons of lengths up to 3.5 m at the same time. The development and the applicability of the vehicle and the large magnet were shown together with the experimental data. The measuring speed has been increased by several magnitudes which allow testing without severe disruption to traffic.
LINEAR POLARIZATION Method can be viable to obtain general condition assessment of strand in structure but is unable to address critical aspect of localization of damage.			
Ali and Maddocks 2003	GHD Company	Small specimens	Enables rapid corrosion rate measurement and is useful for measuring low rates of corrosion, less than 2.5 microns/year. The drawback of the method is that it assumes that all individual wires undergo uniform corrosion. Therefore, localized corrosion in the form of pitting is averaged over the entire strand length. Measurement of the polarization resistance in large concrete structures requires further investigation especially in the presence of corrosion mitigation measures that limit current spread in the strand. Additionally, probes cannot distinguish between reinforcement and prestressing steel.
Broomfield 1994			This method is useful for lab testing but some modifications are required for field testing. The existing method has been enhanced by the addition of a sensor controlled ring device.
Mehta and Monteiro 2006			In this method, the corrosion rate of a metal is measured from its polarization resistance. This is determined by applying a small DC voltage and recording the results. This method is widely used both in the field and for research. Table 11-3 in the reference gives average values for resistance, which are correlated to corrosion. The advantage of this method is that commercial equipment is available to measure polarization resistance of existing structures in the field and also the results can be obtained quickly. There are also some limitations to this technique. The whole reinforcing bar is polarized and a single value is determined as the resistance, which is essentially an average result for all steel in the vicinity. The problem here is that it assumes corrosion to be uniform throughout the reinforcement section which is not true for the usual case. Also, the method assumes that concrete resistivity is low whereas it is usually high. Especially in conditions where concrete is dry, this assumption may lead to significant error.

Reference	Sponsoring Agency	Structure Type	Findings/Discussion/Summary
Suh et al. 2007	FLDOT and FHWA	Twenty-two 1\3 scale models of prestressed piles	As part of a long-term experimental program, it was necessary to monitor corrosion inside concrete specimens. In order to decrease the time required to corrode the strands, either a constant current or voltage was applied, or the specimens were initially produced by using permeable concrete with high w/c ratios or by casting specimens with inclusion of chlorides. Three different techniques were employed to monitor corrosion and determine the effectiveness of several FRP materials on corrosion prevention. Half-cell potential measurement, linear polarization measurements and soil resistance. Although unexpected fluctuations were obtained, they are useful for approximate results and in understanding the general trend and increase in corrosion. The main focus of the project was to investigate the effects of FRP on mitigation of corrosion rather than testing these non-destructive techniques for accuracy.
ELECTRICAL RESISTANCE Similar to linear polarization, this method has limited usefulness *in situ*.			
Ali and Maddocks 2003	GHD Company		Electrical resistance of the steel strands is being used to detect corrosion. Since resistivity is constant for a given steel sample, change in electrical resistivity must be caused by a change in cross-section, most probably due to corrosion. As the section gets smaller, electrical resistivity increases. It is assumed that all wires undergo the same amount of corrosion. A drawback of the method is that short circuits at the anchorage points or within the structure (stirrups) affect results.
Mehta and Monteiro 2006		Concrete surface	The resistivity of concrete when exposed to an electrical current is highly correlated to the amount of ions in the concrete and the state of the reinforcement. Highly resistive concrete has little possibility of corrosion. Therefore it is possible to assess the degree corrosion by determining the resistivity of the section. CEB-192 (Comite Euro-International du Beton) has estimated values for concrete resistivity depending on the amount of corrosion rate.
SURFACE POTENTIAL SURVEY/HALF-CELL POTENTIAL SURVEY Well-established ***STANDARDIZED*** technique. While cumbersome, is presently most viable and widely used *in situ*.			
Ali and Maddocks 2003	GHD Company		Half-cell or electrode potential mapping is a widely accepted method for detecting corrosion of steel embedded in concrete. The entire surface is mapped by recording the surface potentials with respect to a reference electrode. Locations with higher negative potentials indicate areas of corrosion. However, this correlation does not always hold and surface potentials are greatly influenced by surface conditions. It is not possible to distinguish between reinforcement and prestressing strands.

Reference	Sponsoring Agency	Structure Type	Findings/Discussion/Summary
Mehta and Monteiro 2006		Concrete surface	The corrosion potential of the steel in concrete can be measured as the voltage difference between the reinforcement and the reference electrode in contact with the concrete surface. Measured potentials have been assigned to certain corrosion probabilities in ASTM C 876. By moving the reference electrode, a relative potential map can be made which shows areas that are more susceptible to corrosion. This is a quick and inexpensive method that may be used during planning of areas that need repair. However, results may be affected by the degree of humidity of concrete, oxygen content near the reinforcement, existence and extent of microcracks, or electrical stray currents. Due to these reasons, ASTM has specified certain conditions where the technique should not be applied. It should be recognized that results of this method are not quantitative. Connection of the equipment to the steel reinforcement is required.
Suh et al. 2007	FLDOT and FHWA	Twenty-two 1\3 scale models of prestressed piles	As part of a long-term experimental program, it was necessary to monitor corrosion inside concrete specimens. In order to decrease the time required to corrode the strands, either a constant current or voltage was applied, or the specimens were initially produced by using permeable concrete with high w/c ratios or by casting specimens with inclusion of chlorides. Three different techniques were employed to monitor corrosion and determine the effectiveness of several FRP materials on corrosion prevention. Half-cell potential measurement, linear polarization measurements and soil resistance. Although unexpected fluctuations were obtained, they are useful for approximate results and in understanding the general trend and increase in corrosion. The main focus of the project was to investigate the effects of FRP on mitigation of corrosion rather than testing these non-destructive techniques for accuracy.
ELECTRICAL TIME DOMAIN REFLECTOMETRY Useful for detecting voids in grouted tendons although provides little other useful data.			
Okanla et al. 1997		Post-tensioned structure	The method is used to detect, locate and identify the sizes of voids filled with air or water in post-tensioned ducts. Is not used for corrosion assessment, although results of ETDR can be used to direct further investigation.
FIBER OPTIC SENSORS Demonstrated to be reliable and durable method for assessing displacement or strain. Hardware is not inexpensive at this time.			
Hariri and Budelmann 2001		Exposed strand	The basic idea of the stranded optical fiber sensor is to measure the degree of attenuation of light due to microbending. In a wire, as the strain or stress increases, so does the amount of attenuation. Analysis of the ratio of light input to output when compared to a reference wire enables the determination of internal stress levels of the wire.
MAGNETO-ELASTIC Method requires extensive calibration and is not yet viable for *in situ* application.			
Hariri and Budelmann 2001		Exposed strand	The magnetoelastic measurements correlate the magnetic flux density at saturation of the strand to its stress level. A formulation is given to predict stress, however, the constants need to be calibrated before testing.

Reference	Sponsoring Agency	Structure Type	Findings/Discussion/Summary
NEW PROTOTYPE INSTRUMENT FOR MEASURING REMAINING PRESTRESS Although a promising tool, it is not known if further development or commercialization has been undertaken. Method is most appropriate for QC/QA of spliced repairs.			
Civjan et al. 1998	TXDOT	Exposed strand	A prototype instrument has been developed to estimate the remaining prestress in the strands. A lateral force is applied on the strands and the corresponding lateral displacement is measured. During experimental testing of the research project, the instrument was calibrated for 0.5 in. 7-wire strand with exposed lengths of 1.5 to 3.75 ft. Overall, the prototype instrument for estimating stress levels in prestressing strands performed very well. It was seen that with the use of this device, measurements can be conducted quickly and inexpensively. Furthermore, the test results were within 10% of actual stress levels. With this method, the induced prestress during repair works such as strand splicing or preloading can also be monitored and checked for adequacy. However, further experimentation with different types of strands, number of wires and exposure lengths is necessary before field application since all of these factors may affect results. The suitable calibration charts must be present before actual testing.
FLAT-JACK METHOD This method is partially destructive and may be complicated to apply *in situ*. Nonetheless, the method may be appropriate in certain circumstances.			
Turker 2003	PITA, Lehigh U.	Concrete structures	The distance between two points on the surface is precisely measured. Afterwards, a slit is cut into the concrete in between these two points in a direction perpendicular to the stress to be determined. The local stress in the structure is relieved due to the slit and so the relative distance between the two points decreases, for the case of compression. Then, a hydraulic jack is inserted into the slit and pressure is applied until the distance between the two points is equal to the initial distance. The pressure on the jack is equal to the internal stress in the structure. The advantage of this method is that the elastic modulus of concrete is not used to measure the in-situ stress. The disadvantage is that the stress distribution applied by the jack might be different than the internal stress, which might lead to an error.
NEBRASKA METHOD Method has limited application and calibration is not certain.			
Turker 2003	PITA, Lehigh U.	Girder soffits	The method was developed to measure the effective prestress in prestressed concrete bridge girders. A cylindrical hole having a diameter of 1 in. is drilled into the concrete and then a crack is induced in the hole. This crack extends in the direction parallel to the main axis of stress. Then, external force is applied perpendicular to the direction of the crack and the stress necessary to close the crack is determined. This value is then related to the effective prestress. Although special hardware has been developed to clamp to the underside of the bridge girders, it may still be difficult to apply this method in situations where the geometry does not allow it. So, its applicability is limited in this sense.

Reference	Sponsoring Agency	Structure Type	Findings/Discussion/Summary
HOLE- DRILLING STRAIN GAGE METHOD This approach is currently impractical for field application.			
Turker 2003	PITA, Lehigh U.	Isotropic, linearly elastic and homogeneous materials	This is a widely used method to determine the residual stresses near the surface of isotropic, linearly elastic materials. It has been standardized by ASTM E837. A small diameter hole is drilled in an elastic material to relieve strains which are measured by an attached three-gage rosette. A hole is drilled in the center of the strain rosette either completely through the section or to a depth exceeding 40% of the diameter of the strain gage. The method is reported to give good results when the material is isotropic, the internal stresses does not vary greatly, and when the structure is more like a large plate compared to hole size. However, the method is not applicable to concrete due to its heterogeneous nature. The theoretical solution of the problem is presented in detail in the thesis, but experimental verification is needed before application. Known experimental application as yielded marginal results.
ELECTRO-CHEMICAL IMPEDANCE SPECTROSCOPY (AC IMPEDANCE) This approach is currently impractical for field application.			
Mehta and Monteiro 2006			This method measures the polarization resistance as well as assessing the physical processes in concrete and concrete-steel interface. Small amplitude AC signals are introduced into the concrete and the time lag and the phase response are investigated in the response. Although this method has been used in laboratories, its use is limited for field applications. The necessary equipment is bulky and complex. Similar to the polarization resistance technique, the results are average values over an area and therefore assume uniform corrosion of reinforcement, which is rare. Also, physical connection to the reinforcement is required which might require some amount of concrete removal and patching work. Although this is a powerful method being able to separate the individual processes at different layers, it is still too complicated for practical use.

TASK 3: REVIEW OF REPAIR, REHABILITATION AND RETROFIT TECHNIQUES

A review of repair, rehabilitation and retrofit techniques was carried out. Existing reviews conducted as part of the NCHRP 12-21 project (Shanafelt and Horn 1980 and 1985) and reiterated by Feldman et al. (1996) are summarized in Section 3.1. The fundamental state-of-the-art and state-of-practice has changed very little since these reports were produced. Section 3.2 focuses on developments established ***since*** the NCHRP 12-21 study. Finally, Section 3.3 provides a brief summary of concrete patching techniques.

3.1 Review of NCHRP 12-21

NCHRP Report 226 (Shanafelt and Horn 1980) focuses on providing guidance for the assessment, inspection and repair of damaged prestressed concrete bridge girders. Suggestions are given for standardized inspection including proper techniques, tools and forms. The authors emphasize the need to separate the damage assessment tasks (inspection) from the engineering assessment tasks (load rating, etc.).

Often, the decision to replace or the repair method chosen is not appropriate for the level of damage, resulting in inefficient and improper repair actions. A damage classification system, allowing users to quantify the damage present is proposed. Damage is classified into one of three categories:
MINOR damage is defined as concrete with shallow spalls, nicks and cracks, scrapes and some efflorescence, rust or water stains. Damage at this level does not affect member capacity. Repairs are for aesthetic or preventative purposes.
MODERATE damage includes larger cracks and sufficient spalling or loss of concrete to expose strands. Moderate damage does not affect member capacity. Repairs are intended to prevent further deterioration.
SEVERE damage is any damage requiring structural repairs. Typical damage at this level includes significant cracking and spalling, corrosion and exposed and broken strands.

Minor and moderate damage can be repaired via patching and painting; guidelines are provided for these tasks. Since minor and moderate damage do not require structural repairs, emphasis is placed on severe damage.

In *Report 226*, eleven different repair methods were developed for the severe damage condition and are discussed in detail; none however was demonstrated or tested. Each repair technique was evaluated to provide an overview of the processes and advantages and limitations of the method. Guidelines were proposed based on service load capacity, ultimate load capacity, overload capacity, fatigue life, durability, cost, user inconvenience and speed of repairs, aesthetics and range of applicability. Evaluation of the repair techniques based on these parameters was conducted using a value-engineering process. Areas to be considered for future research were identified, particularly associated with the proposed splice repairs. Some of the repair techniques presented needed to be tested and evaluated for strength and fatigue loading.

Repair methods considered in *Report 226* were external post-tensioning, metal sleeve splicing (to avoid confusion, this method will be referred to as 'steel jacketing' in the present work), strand splicing, a combination of these methods, and replacement.

External post-tensioning is affected 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). The steel rods, strands or bars are then tensioned by jacking against the bolster or preload (which will be discussed later). Examples of this method are shown in Figure 3.1. Splice 1 (*Report 226* designation) used Grade 40 reinforcing bars, Splice 2 used Grade 60 steel rods encased in PVC conduits as a corrosion resisting measure, and Splice 4 used a corbel that was continuous over the entire length of the girder for corrosion protection of six post-tensioned 270 ksi strands. Post-tensioning force in the case of Splice 1 is nominal and is induced by preload only. Today, Splice 2 details would generally be accomplished using high strength (150 ksi) post-tensioning bars (such as Williams or Dwyidag products). In this case, post-tensioning force may be induced by jacking or preload or a combination of both. For Splice 4, post-tensioning force will typically be induced by jacking. An advantage of Splice 4 is that it can also be designed as a 'harped' system, affecting greater efficiency, particularly with respect to restoring excessive vertical deflection of the girder. In this case, both bolsters and deviators must be attached to the beam.

Design of external post-tensioned repair systems is relatively straight forward using a simple plane sections analysis (recognizing that the post-tensioning bar is unbonded). The attachment/interface of the bolsters, however, requires significant attention. These elements are 'disturbed regions' subject to large concentrated compression forces. Additionally, sufficient shear capacity along the interface between the bolster and existing beam must be provided to transfer the post-tensioning force. Effective shear transfer often requires the bolsters themselves to be post-tensioned (transversely) to the girder to affect adequate 'friction' forces along the interface. Finally, the design of the bolsters and interface must consider the moments induced by the eccentric post-tensioning forces.

Steel jacketing is the use of steel plates to encase the girder to restore girder strength. With this repair technique, post-tensioning force can only be introduced by preloading. Splice 3, shown in Figure 3.2, employs a steel jacket. Generally, this method of repair will also require shear heads, studs or through bars to affect shear transfer between the steel jacket and substrate beam. 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' end of beam in most applications). Additionally, the jacket will need to be grouted in order to make up for dimensional discrepancies along the beam length. Neither of these details is addressed in *Report 226*.

Strand splices are designed to reconnect severed strands. Methods of reintroducing prestress force into the spliced strand are preloading, strand heating and torquing the splice; the latter is most common, essentially making the splice a turnbuckle of sorts. Strand heating is a method whereby the strand is heated, the strand splice is secured to the strand and as the strand is allowed to cool, it shrinks, thus introducing tension back into the strand. Strand heating of conventional high-strength prestressing strand is not believed to be a terribly rational method of affecting any reasonable prestrain: either a) a long length of strand must be heated; or b) a short length of strand must be heated to a high temperature. The former is impractical in a bridge girder and the latter will affect the material properties of the strand. ***Strand heating is not recommended***. Commercially available strand splices have couplers connected to reverse threaded anchors; as the coupler is turned, both anchors are drawn toward each other, inducing a prestress in the attached strand (see Figure 3.3). Schematic examples of strand splices are shown in Figure 3.4. Splice 6 utilizes strand chucks to splice the strands and strand heating to induce tension (recall that the methods reported in *Report 226* were not tested in relation to this work). Splice 7 uses a strand splice that has a coupler nut in the middle which is tightened to reconnect and introduce tension into the strand. Splice 8 uses a round steel bar which connects to a steel transfer plate and then to the strands to reconnect the strands.

Repair techniques may be combined. Combination of repair techniques will allow the user to employ the advantages of each repair. For example, Splice 5, shown in Figure 3.5, uses post-tensioning in conjunction with steel jacketing to restore girder strength. The post-tensioning addresses girder serviceability while the steel jacket reinforces the girder's ultimate capacity.

Most repairs proposed in *Report 226* make use of preloading during girder repair. ***Preload*** is the temporary application of a vertical load to the girder during the repair. The preload is provided by either vertical jacking or a loaded vehicle. If the damage has caused a loss of concrete without severing strands, preloading during concrete restoration can restore the strength of the girder without adding prestress. Because preloading may be used to restore partial or full prestress to the repaired area, it effectively reduces tension in the repaired area during live load applications. It is for this reason that preloading is suggested for most repairs, particularly those including patching. 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.

It must be noted that Shanafelt and Horn, in *Report 226*, addressed relatively small prestressed elements having only 16 strands. In this case, the preload required to affect the post-tensioning force is relatively small. In this case the structural system is similar in scale to a parking garage. As elements become larger – as for a bridge – the level of preload required becomes very large and not practical to apply. The effectiveness of considering preload is improved with reduced dead-to-live load ratios; however these are not typical in concrete structures.

NCHRP *Report 226* provides the selection matrix, shown in Table 3.1, for selecting repair methods for prestressed girders. Guidelines presented for each repair method are as follows. The 'number of strands' that may be spliced must be placed in context. The prototype girders considered in this study only had 16 strands.
External Post-tensioning: replacing the loss of more than 6-8 strands may be difficult, but this method can be used to restore strength and durability to damaged girders and add strength to existing bridges.
Strand Splicing: this method is good for repair of a few strands but is limited by the geometry of the strand splice and concrete cover.
Steel Jacketing: this method was successfully used to replace the loss of 6 strands, but is not very common.

The second phase of the NCHRP 12-21 project and the focus of NCHRP *Report 280* (Shanafelt and Horn 1985) was to provide a practical user's manual for the evaluation and repair of damaged prestressed concrete bridge members. Significantly, some of the repair methods presented in the earlier *Report 226* were load tested and suggestions for their implementation are given. It is important to note that the girders were never loaded to their ultimate capacity. All tests were conducted on a single girder with artificial damage and one of the repair techniques applied.

Ten different load tests were conducted on a single I-girder to measure the behavior of each repair:

1. Load girder up to 75% of the calculated ultimate load capacity;
2. Add concrete corbels and post-tension high-strength bars and load;
3. Disconnect high-strength bars and load (same as load test 1 but girder is now cracked);
4. Break out specified concrete to sever 4 strands (25% of the total 16 strands) and load,
5. Splice 4 strands with single strand splice and patch and load;

6. Reconnect post-tension high-strength bars (same test as test 5 but with external post tensioning);
7. Disconnect bars, break out concrete and sever the four strands spliced in test 5 and load;
8. Patch the girder and tension the external bars;
9. Disconnect bars, break out patch, sever 2 more strands for a total of 6 and splice them with a steel jacket and load; and
10. Load the steel jacketed girder to 100% of the calculated ultimate moment capacity.

While the tests of each repair technique generally demonstrated a sound response, the facts that a) there was no control specimen with which to compare results; and b) the repairs were sequential and thus the degree of damage was necessarily incremented between tests affected the ability to draw conclusions from this test program. Although a significant amount of test data is provided, few conclusions are or can be drawn.

3.2 Review of Studies of Repair Techniques

A review of repair techniques for prestressed concrete bridge girders was undertaken. The review focused on the state-of-the-art established since the completion of NCHRP 12-21 in 1985. The following table summarizes this review.

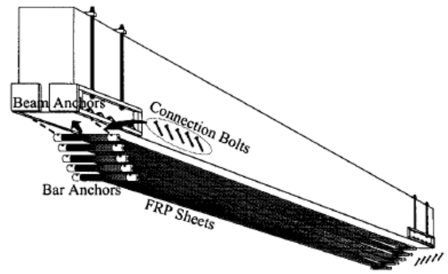
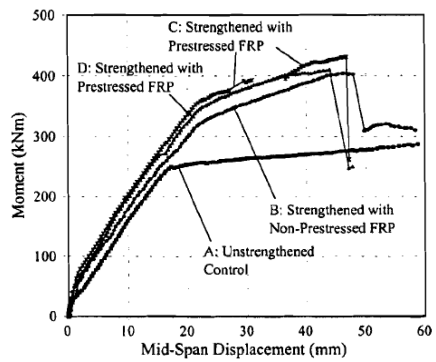
CITATION	SPONSORING AGENCY	SUMMARY AND NOTES
PRESTRESSED REPAIR BASED ON METHODS DESCRIBED IN NCHRP 12-21 (i.e.: 'traditional methods') These studies address those repair methods described in NCHRP 12-21. In some cases, the cited work represents the only known reported experimental applications of the cited repair methods. All of these studies were founded on the work presented in the NCHRP 12-21 project.		
Preston et al. 1987	PennDOT and FHWA	This report proposed design repairs for adjacent prestressed concrete bridge members. The 2 repairs analyzed were a) 6-#6 epoxy coated reinforcing bars suspended 1" below the soffit and then covered with concrete creating a total depth of 3"; and b) four post-tensioned 0.5" diameter, epoxy coated, low-relaxation strands installed 2" (with cover total depth of repair was 3") below the soffit of the beam and anchored at a tension strength of 21.5 kips each. These repairs performed well enough. However, they decreased the clearance of the structure.
Olson et al. 1992	Minnesota DOT and NSF	The goal of this report was to determine the effective prestress in the strands after 20 years, to determine the influence of impact damage on girder performance, to evaluate the performance of two impact damage repair schemes under static, fatigue, and ultimate loadings, and to develop a model to estimate the strand stress ranges in damaged girders. Appendices with example calculations are available for the previously listed goals. The repairs were 2 examples taken from NCHRP Report 280, but expanded upon – strand splicing and post-tensioning where the girder has only been damaged on one side (to simulate a more realistic damage pattern). Also stated is that the prestress loss is much greater than the calculated expectant losses.
Feldman et al. 1996	Texas DOT	This report focused exclusively on impact damage of prestressed concrete bridge girders and repair methods. Many suggestions for good engineering practice for repair: for example, preloading, material type and reporting practices. The scope of this report included documenting current practices in the repair of prestressed concrete girders. From survey data, it is common that little repair work is done for minor damage, patching is typical for moderate damage and often times a girder is replaced in cases of severe damage. This work is very similar in nature to the NCHRP 12-21 reports, but includes current state of practice.
Labia et al. 1996	NSF	The primary objective of this report was to determine strength and serviceability of 20 year old girders. Another objective was to determine the behavior of the member during static and dynamic loading and a strand repair method applicable in a box girder was sought. Two girders were tested statically and dynamically to see how they would respond after being taken out of service. One girder was repaired with Grabb-it strand splices. This is a very good example with a good explanation of using strand splices. The splice performed satisfactorily (both girders performed comparably and were close to capacities calculated from the code). The girder with the Grabb-it splice worked well and should be a viable, cost-effective repair method (it has been used in California, Washington and Oregon). It is important to note that the authors preferred to use the "turn of the nut" method as compared to the calibrated torque measurement as suggested by the splice manufacturer.

CITATION	SPONSORING AGENCY	SUMMARY AND NOTES
Zobel and Jirsa 1998	Texas DOT and FWHA	The goal of this study was to investigate the performance of strand splice repair in prestressed beams. Parameters considered are overall structural performance and ease and reliability of splice installation. All Splices were weaker than the strength of the strand. “Internal strand splices should not be used for unbounded applications where slippage can occur at the anchorages.” Interestingly, the splices are only recommended in two specific instances: 1. If ultimate flexural strength of the girder with the remaining undamaged strands is greater than the factored design moment, then repair by internal strand splices could be used to reduce the range of stress imposed on the other strands and 2. If fatigue is not a major concern, internal splice methods could be used to restore ultimate flexural strength to a damaged girder. In any case, repairing more than 10-15% of the total number of strands within a single girder is not recommended. All splices gave a minimum strength of 85%of the nominal strength of the strand.
Tabatabi et al. 2004	Wisconsin DOT	This report was unique in that the repairs were focused on beam ends (within the last two feet of the end). The beam-ends were subjected to wet/dry cycles of salt-water sprays together with imposition of an impressed electric current to accelerate the corrosion process. After an initial exposure period of 6 months, most of the previously untreated beam-ends were also repaired/protected and one was left unprotected. The corrosion process was allowed to continue for a total exposure time of 1½ years. Table 33 shows how many side girders and bottom girders were affected by corrosion as well as providing an overall rating for the amount of strands affected. “Surface treatments, while reasonably effective over the short-term, have demonstrated limited effectiveness over the long term, unless they are applied prior to chloride contamination. Cathodic protection, while effective, is not commonly employed due to the high component and maintenance costs as well as the complexity of the method. In addition, due to the possibility of hydrogen embrittlement, cathodic protection of prestressed concrete beams is generally not recommended. Research studies have established the effectiveness of FRP composites to prevent and mitigate corrosion-damage in concrete columns.” Lastly, Table 34 compares the beam end ratings with the overall rating for each sample to display the results at a glance.
EXTERNAL, NON PT CFRP RETROFIT TECHNIQUE (CFRP) In this repair method, CFRP materials are applied to the concrete member as tension reinforcement. The CFRP is applied using a structural adhesive and is not stressed in any way prior to application. Thus the CFRP is composite with the substrate concrete only in resisting loads applied following CFRP application. For this reason, non-PT retrofits are typically only used to affect the ultimate capacity of the prestressed concrete member. Typically, the critical limit state of such externally bonded retrofit measures is associated with failure of the CFRP- concrete interface; this is conventionally referred to as ‘bond’ failure.		
Russo et al. 2000 **CASE STUDY**	Iowa DOT and Iowa Highway Research Board	Two bridges in IA were tested in situ. One was damaged by impact while the other was not damaged and thus used as a reference. After testing of the bridges, one damaged beam was tested in "as-is" condition while the other was tested with a CFRP repair. Also, a 3-beam laboratory bridge model was tested. After testing two beams were removed and tested individually (undamaged and intentionally damaged with CFRP repair). Results are briefly discussed for all tests. Paper was a shortened sample of IA DOT Project HR-397.
Tumialan et al. 2001 **CASE STUDY**	Missouri DOT and UMR UTC	An impact damaged girder is repaired with CFRP to help restore flexural capacity. The CFRP repair restored a flexural capacity of 190 k-ft. Good explanations of the use of CFRP. The entire application process is explained. Detailed calculations are given in the appendix! A construction error occurred resulting in a “blister” in the CFRP sheet. The CFRP “blister” repair was also discussed.

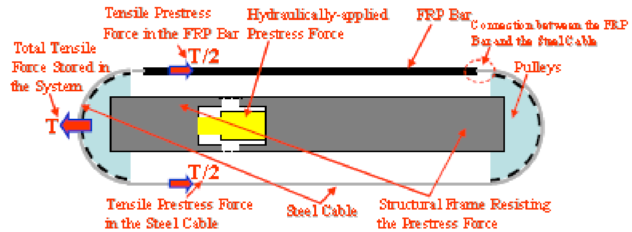
CITATION	SPONSORING AGENCY	SUMMARY AND NOTES
Schiebel et al. 2001 **CASE STUDY**	Missouri DOT	An impact damaged girder is repaired with CFRP to help restore flexural capacity. The CFRP repair restored a flexural capacity of 187 k-ft. Good explanations of the use of CFRP. The entire application process is explained. Detailed calculations are given in the appendix! Pull off and torsional tests were conducted. Additionally, UMR was also contracted to study long term bond behavior over the next 5 years, but it is not presented in this report.
Klaiber et al. 2003 **CASE STUDY**	Iowa DOT and Iowa Highway Research Board	A field application of CFRP flexural repair is presented in this paper. Deflections of damaged and repaired members are compared. Based upon this study: Flexural strengthening of impact damaged P/C girders is feasible when approximately 15 percent of the strands are severed; CFRP sheets restore a portion of the flexural strength lost when P/C girders are damaged; Transverse CFRP sheets assist in the development of the longitudinal CFRP plates and prevent debonding. Such jackets also confine patch material; CFRP reduced beam deflections in some cases by as much as 20%. Actual deflections measured, however, were very small.
Wipf et al. 2004	Iowa DOT and Iowa Highway Research Board	Similar to other reports discussed here: Impact damage repaired with CFRP. One unique issue that was addressed was confinement of the patch. Since there was significant loss of concrete section, the patch was confined by CFRP strips which helped to hold the patch in place during loading. Another interesting point discussed fatigue of the remaining prestressing strands in a damaged girder. Fatigue in these strands is not an issue as long as the damaged girder has not developed flexural cracks. If flexural cracks have developed, the fatigue life of the remaining prestressing strands needs to be evaluated.
Green et al. 2004	Florida DOT	Due to the increasing number of overheight vehicle impact damage, FL decided to setup a research program to test various FRP repairs. For the six tests, different strengthening procedures were used: one unstrengthened and undamaged girder as a control, one purposely damaged girder as a control and the remaining four purposely damaged girders repaired by: a wet layup procedure, pre-impregnated fabric resin, spray application and another wet layup procedure (from another manufacturer). The installation of each procedure is described including limitations. For the various repairs, the experimental and theoretical capacities achieved were in the range of 91-108% and 96-114% of the test girder, respectively. At a minimum, deflections were reduced by 23% and strains at the bottom centerline of the girder were reduced by 39%.
Reed and Peterman 2004	Kansas DOT and UMR UTC	This investigation has shown that both the flexural and shear capacities of 30-year-old damaged prestressed concrete bridge girders (in this case T-beams) may be substantially increased with CFRP sheets. The analytical model based on strain compatibility closely approximated both the stiffness and flexural capacity of the two strengthened flexural specimens. CFRP stirrups also increased the shear capacity of the specimens tested, but did not change the failure mode of the specimens.
Reed and Peterman 2005	Kansas DOT	This report was conducted on decommissioned T-beams. Previous damage of the beams was recorded and repairs began. Similar to the FDOT report in explaining patching and epoxy injection of cracks before CFRP repair. CFRP's were placed with the wet lay-up process. The ultimate capacity of these specimens was increased by over 20% of the actual strength of the base specimen, which corresponds to over 45% of the original design capacity. However, the increase in stiffness was rather small when considering the large increase in flexural capacity. FRP U-stirrups were successful in overcoming horizontal shear failures. Lastly, flexural analysis conducted by the strain compatibility theorem and moment curvature provided results that correlate well with experimental data.

CITATION	SPONSORING AGENCY	SUMMARY AND NOTES
Reed et al. 2007	Kansas DOT and UMR UTC	The goal of this study was to investigate the feasibility of CFRP repairs (both sheet and NSMR applications) on an older (30-yr old) structure. While it was found that good bond strength was achieved (meaning CFRP repairs are feasible) results for epoxy injection repairs varied. In all but one case, epoxy injections restored original stiffness. Surface preparation and techniques presented in this paper ensured a good quality bond between concrete and CFRP sheet. Similarly, surface preparation and techniques presented in this paper ensured a good quality bond between NSMR and concrete.
Aidoo, J. (2004)	US DOT and FHWA	Eight full-scale reinforced concrete bridges were retrofitted with three different FRP systems with a goal of investigating midspan FRP debonding. The CFRP systems used were conventional adhesive applied (externally bonded strip), NSM and Powder Actuated Fastener (PAF). All three strengthening systems were successful in restoring flexural capacity to allow the girders to carry HS25 loads. Midspan debonding failure can be predicted using the intermediate crack induced debonding models provided they account for the ratio of FRP plate to substrate width and loading and specimen geometry.
Aidoo et al. 2006	US DOT and FHWA	Eight full-scale reinforced concrete bridges were retrofitted with three different FRP systems. Specimens were subjected to monotonic loading to failure with and without significant fatigue conditioning. Crack induced debonding was observed to be the dominant failure mode for monotonically loaded beams. Fatigue conditioning caused degradation of the CFRP-to-concrete interface. It was found that the current practice and code at the time was sufficiently conservative for monotonic loading, but it is suggested here that a strain reduction should be included for fatigue loading.
Quattlebaum et al. 2005	South Carolina DOT, FHWA and the NSF	A comparison of three CFRP applications is conducted in this study; the methods are conventionally applied adhesive, powder actuated fasteners and NSM reinforcement. Ten medium-scale beams were tested (six under fatigue loads and four under monotonic loading). All methods were found to increase beam capacity under monotonic loads. However, it was found that the conventionally applied adhesive performed the best out of the three methods.
Ramanathan and Harries 2008		Debonding of CFRP at the CFRP-to-concrete is often the failure limit for externally bonded repairs. With this in mind, the study was conducted to determine the affect of FRP width-to-substrate width on bond performance by evaluation FRP strain and debonding. For such flexural retrofits, load carrying capacity increased and displacement ductility decreased with increasing CFRP reinforcement. When all retrofit measures at a particular applied load level are compared, it was found that a 'larger number of smaller strips' exhibited preferable debonding behavior.
Harries et al. 2006	South Carolina DOT and FHWA	Characteristics of FRP-to-concrete bond are studied. It was found that the effects of fatigue on bond deterioration must be accounted for, even when small fatigue loads are present. Debonding was noted at CFRP strains significantly lower than that specified for monotonic loading. The performance of the CFRP under fatigue was found to be in a stress range of only about 4% of the material capacity, suggesting that there is little endurance limit for CFRP under fatigue loading.

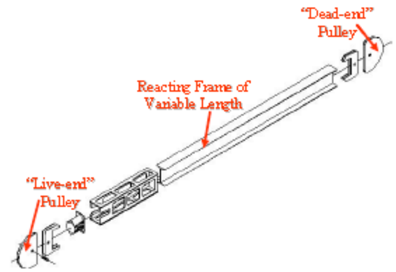
CITATION	SPONSORING AGENCY	SUMMARY AND NOTES
EXTERNAL, PRESTRESSED OR POST-TENSIONED CFRP RETROFIT TECHNIQUE (PCFRP) Prestressed or post-tensioned (the terms are used inconsistently in the literature) CFRP repairs require affecting a tensile stress in the CFRP. Under stress, the CFRP is attached to the prestressed member. There are three approaches to installing PCFRP systems. The following terminology is adopted: ***prestressed CFRP***: The CFRP is drawn into tension using external reaction hardware and is applied to the concrete substrate while under stress. The stress in 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 potentially susceptible to large losses at stress transfer and long term losses due to creep of the adhesive system. Additionally, details (such as FRP U-wraps) must be provided to mitigate debonding at the termination of the CFRP strips or plates. Prestressed CFRP systems are analogous to prestressed concrete systems where the stress is transferred by bond to the structural member. ***unbonded post-tensioned CFRP****:* 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. 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 is a consideration. Such systems must be designed with sufficient clearance between the CFRP and substrate concrete to mitigate the potential for fretting. Unbonded post-tensioned systems are analogous to conventional unbonded post tensioning systems. ***bonded post-tensioned CFRP***: The CFRP is stressed and anchored in the same fashion as unbounded systems. Following anchorage, the CFRP is bonded to the concrete substrate resulting in a composite system with respect to loads applied following CFRP anchorage. Since the adhesive system is not under stress due to the post-tension force, adhesive creep is not a significant consideration with this system. The bonding of the CFRP may help to mitigate creep losses associated with the anchorage. Bonded post-tensioned systems are analogous to conventional bonded post tensioning systems. All PCFRP systems impart some prestress to the concrete member. This can reduce initial deflections and relieve the stresses in the existing prestressed or mild reinforcement. PCFRP systems result in more efficient utilization of the CFRP material.		

CITATION	SPONSORING AGENCY	SUMMARY AND NOTES
Wight et al. 2001	Canada MOD and NSERC	This paper reports a bonded post-tensioned CFRP repair technique. CFRP reinforcement is tensioned against the beam directly and then bonded to the beam. It is found that prestressing the sheet greatly increases cracking load and ultimate strength. Additionally, anchored PCFRP sheets performed better than their non-anchored counterparts.  Proposed prestressing system anchored by beam.  Moment-displacement plot comparing unretrfitted member with non-prestressed and prestressd FRP repairs.

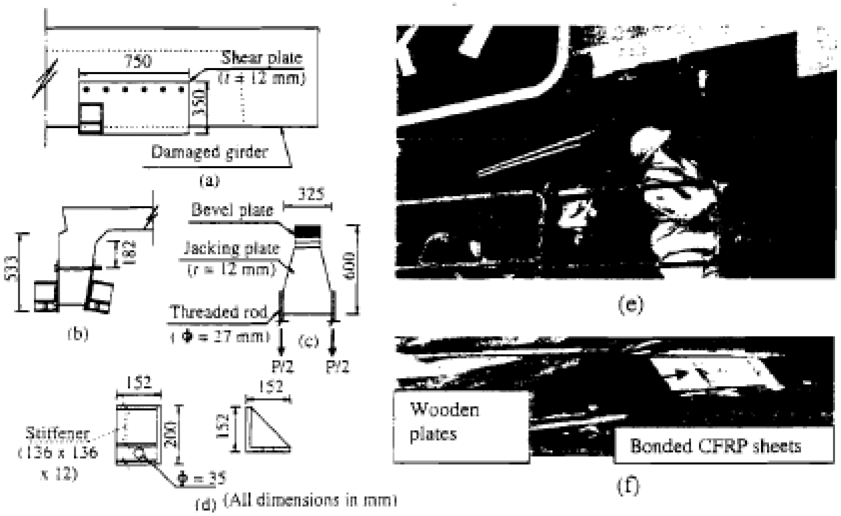
<table>
<tr><th>CITATION</th><th>SPONSORING AGENCY</th><th>SUMMARY AND NOTES</th></tr>
<tr><td>Casadei et al. 2006</td><td>UMR UTC and NSF RB2C.</td><td>Three prestressed concrete I shaped girders were used in this report; one control beam, one damaged beam strengthened with prestressed CFRP sheets and one damaged beam strengthened with a PNSM repair. Both repairs restored ultimate strength of the girder but the PNSM beam restored the original action as well. The PCFRP sheet repaired girder had cracks resulting from biaxial bending since one side of the girder was stronger than the other. A proprietary external prestressing frame was used to stress the CFRP.

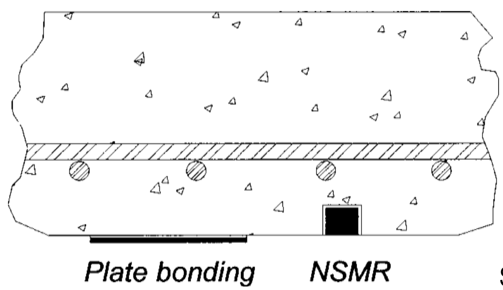


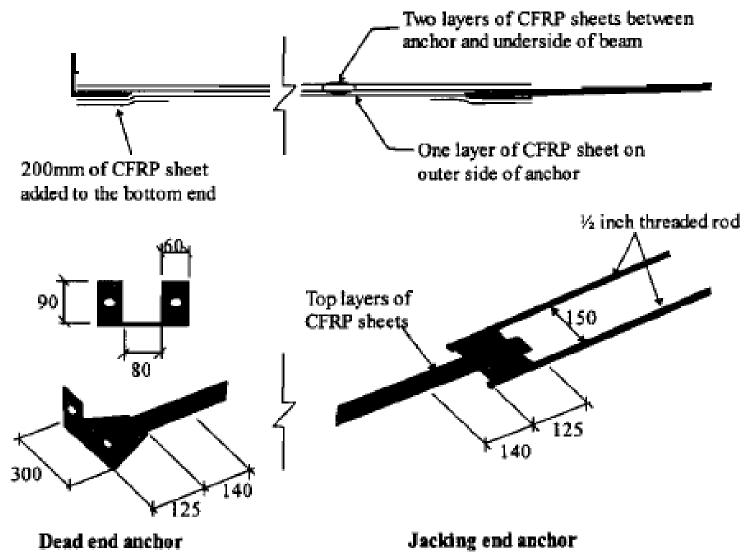
Schematic of prestressing system

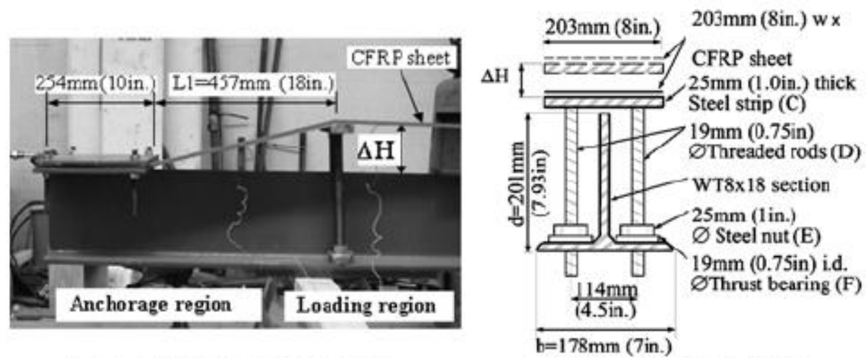


Prestressing system used on beam soffit</td></tr>
<tr><td>Kim et al. 2007.
ANALYTICAL</td><td>ISIS Canada, NSERC and Queen's University</td><td>This paper describes the application of a fracture mechanics model to concrete structures including strengthening with PCFRP. Evaluation of this model was done by comparison to results from finite element analysis, strength-based model, conventional design model and experiments.</td></tr>
<tr><td>Aram et al. 2008</td><td>EMPA</td><td>Beams were strengthened with PCFRP sheets, but had different 'anchorages'. One beam was strengthened with non-PT CFRP strips and two beams were strengthened using prestressed CFRP strips. An external reaction frame that employed the 'gradient method for anchorage' was used. With this method the prestress force is gradually reduced to zero at its ends by means of a protocol of incremental stressing, adhesive application and stress release. It was determined that prestressing the strip had no advantages (from both serviceability and load perspectives)</td></tr>
<tr><td>Yu et al. 2008b</td><td>NSF RB2C</td><td>Theoretical and experimental results related to prestressing levels that can lead to debonding (and concrete cover separation) immediately after transfer and during flexural loading are discussed in conjunction with the investigation on the increase in flexural capacity with the use of CFRP U-wrap end anchors. Concrete cover separation was seen in the prestressed sheet applications. This failure was averted when CFRP U-wraps were employed.</td></tr>
<tr><td>Kim et al. 2008b</td><td>ISIS Canada, NSERC and Queen's University</td><td>The authors indicated that a beam strengthened with PCFRP sheets will fail prematurely and lower than even the CFRP repaired structure when there is a lack of proper anchorage. Ten specimens (one was a control) were tested with various anchorage systems to determine the strength of each system. In all cases, it was found that the prestress loss was less than 1% and the prestress force was successfully transferred into the beam. Length of the side sheets and U-wraps should be considered as part of the designed according to the shear stress variations along the loading span.</td></tr>
</table>

CITATION	SPONSORING AGENCY	SUMMARY AND NOTES
Kim et al. 2008c **CASE STUDY**	ISIS Canada, NSERC and Manitoba DOT	This experiment was the first application of bonded post-tensioned PCFRP sheets for repair used in N. America. The sheets were stressed directly against the beam in need of repair using a steel bolster arrangement as shown below. The intent of the repair is to restore strength and increase serviceability (via crack control). Assessment of the girder based on AASHTO LRFD validates the effectiveness of the repair.  PCFRP system used *in situ*
Herman, T. (2005) **CASE STUDY**	Structural Preservation Systems	The application of this system on two prestressed concrete box girder bridges is discussed. The intended repair of the prestressed concrete box girders was to restore flexural capacity as well as replace some of the lost prestressing forces; employment of the CarboStress system as the repair technique proved successful at restoring flexural capacity and prestressing force. Additionally, this method saved monetary and material resources and minimized construction time and traffic closures.
El-Hacha and Elbadry (2006)	NSEC and ISIS Canada	The external post-tensioning concept can be completed with CFRP instead of steel as the post tensioning material. The use of post tensioned 7-wire CFRP cables (CFCC) for strengthening of concrete beams was examined here. CFCC have little strength laterally, thus stressing these cables is difficult. As a result, a special alloy was die-cast onto the cable to allow the CFCC to be gripped and stressed. The experiment showed comparable results to steel post-tensioned repairs. The post-tensioning force created a stiffer beam and thus a stiffer load-deflection response.
NEAR-SURFACE MOUNTED NON PT CFRP TECHNIQUE (NSM) Near surface mounting is a variation of external bonding of CFRP. The CFRP is installed in shallow grooves cut into the concrete substrate. NSM has the advantage of significantly improved bond behavior as compared with externally bonded systems.		
Reed et al. 2007	Kansas DOT and UMR UTC	The goal of this study was to investigate the feasibility of CFRP repairs (both sheet and NSMR applications) on an older (30-yr old) structure. While it was found that good bond strength was achieved (meaning CFRP repairs are feasible) results for epoxy injection repairs varied. In all but one case, epoxy injections restored original stiffness.

CITATION	SPONSORING AGENCY	SUMMARY AND NOTES
NEAR-SURFACE MOUNTED PT CFRP TECHNIQUE (PNSM) PNSM are a variation of the bonded post-tensioned PCFRP approach. While offering improved durability and bond performance, stressing and anchoring an NSM in an *in situ* application is a complex operation.		
Nordin et al. 2002 laboratory stressing system not suitable for *in situ* applications	SKANSKA AB, BPE® Systems AB and The Development Fund of the Swedish Construction Industry	NSM CFRP repairs can be prestressed if serviceability is a concern or non prestressed if ultimate capacity is the only design consideration. The advantages of prestressed NSM are presented here. It is noted, however, that prestressing NSM applications is very difficult and has only been demonstrated in laboratory applications using a stressing procedure that is not practical for use in the field. The stressing operation presented is only useful when the ends of the girder are open and available. The prestressed NSM repairs provide similar results as to the prestressed strip repairs, but NSM repairs achieve a better debonding strength (the typical mode of failure for bonded CFRP systems). A test was done with a reference beam, a beam with a nonprestressed NSM CFRP rod and two beams with a prestressed NSM CFRP rod. The PNSM beams had an increase in load almost 70% greater than that of the reference beam and 37% greater than that of the non-prestressed NSM beam.
Nordin and Taljsten 2006	SKANSKA AB, Sto Scandinavia AB, and SBUF	A good discussion of the advantages to using PNSM reinforcement in a general sense.  Schematic of surface mounted and NSM FRP.
Casadei et al. 2006	UMR UTC and NSF RB2C.	Three prestressed concrete I shaped girders were used in this report; one control beam, one damaged beam strengthened with prestressed CFRP sheets and one damaged beam strengthened with a PNSM. Both repairs restored ultimate strength of the girder but the PNSM beam restored the original action as well. The PCFRP sheet repaired girder had cracks resulting from biaxial bending since one side of the girder was stronger than the other. A proprietary external prestressing frame was used to stress the CFRP.

CITATION	SPONSORING AGENCY	SUMMARY AND NOTES
PRESTRESSED/POST-TENSIONED ANCHORAGE DEVELOPMENT (for CFRP) Most studies have used proprietary or 'home-made' anchorage systems. Considerations in anchorage design include: a) minimizing losses during stress transfer to anchor; b) long term losses associated with creep of the anchor hardware or the CFRP anchorage; c) mitigating the potential for galvanic corrosion; and d) minimizing the size of the anchorage while still effectively transferring the stressing force.		
El-Hacha et al. 2003	Canada MOD, NSERC and ISIS Canada	A mechanical anchorage system was developed to directly tension CFRP sheets by jacking against anchors mounted on the repaired beam. A study of the effects of short- and long-term exposure at room and low temperatures was conducted to examine affects of material and prestress changes resulting from temperature. It was found that the most effective anchor shape was the flat plate anchor. Additional reinforcing of the anchor zone allowed for higher failure loads and caused the sheet anchors to fail. Average short-term prestress loss was approximately 16% of the sheet's ultimate strength (33% of the initial applied prestress force) and was mostly due to anchor set. The prestress loss resulting from low-temperature exposure was less than 4% of the sheet's ultimate strength (about 8% of initial applied prestress). Lastly, the average long-term losses after a year was 11.3% and 4.5% of the sheet's ultimate strength (20.8% and 9.4% of initial applied prestress) at room and low temperature exposure, respectively. The predicted long-term losses after 50 years was reported to be approximately 5% of the sheet's ultimate strength (approximately 10.3% of the initial applied prestress) despite the one-year losses that exceeded this value.  Anchorage systems.

CITATION	SPONSORING AGENCY	SUMMARY AND NOTES
Kim et al 2008a	ISIS Canada, NSERC and Queen's University	The objective of this research program was to develop a nonmetallic anchor system. Various non-metallic anchorage types were investigated and the loss of prestress in the CFRP reinforcement was found; including U-wraps, prestressed U-wraps and mechanically anchored U-wraps. The authors indicated that a beam strengthened with PCFRP sheets will fail prematurely and lower than even the CFRP repaired structure when there is a lack of proper anchorage. Ten specimens (one was a control) were tested with various anchorage systems to determine the strength of each system. These test specimens examined the effectiveness of CFRP U-wraps and side sheets as anchors. The results were compared to the control specimen which used a steel end plate for anchorage. The failure of the beam with steel anchors was brittle while the beams strengthened with nonmetallic anchors displayed a psuedoductile failure. Beams with mechanically anchored U-wraps showed almost the same load capacity as the control specimen. Also, the nonmetallic anchors exhibited better stress redistribution. It should be noted that the control specimen uses a different system than the remaining experimental specimens. The control specimen (J-1) uses a bonded post tension system while the remaining experimental specimens use a prestressed CFRP system. (Anchorages considered can be seen in Figure 3.6)
Yu et al. 2008a	NSF RB2C	This paper describes the development of a deflection controlled FRP prestressing device. A stress relationship can be developed based on the length of the sheet and induced deflection. Testing verified that the device was efficient in applying prestress to the sheet for beam repair. The prestressing device was found to adequately prestress sheets to 40% of their ultimate tensile strength (but prestressing forces greater than this were not investigated due to debonding and cover separation failures). Therefore, the device was successful in prestressing CFRP sheets without the use of hydraulic jacks. The addition of non-prestressed and prestressed CFRP sheets increased the crackling load by three to six times over that of an unretrofited control specimen and the yield and ultimate loads by 25%. It should be noted that the use of U-wraps for anchorage of prestressed sheets averted concrete cover separation resulting from the transfer of the prestressing force from the sheet to the girder.  Prestressing system; stressed CFRP is then adhered to beam

CITATION	SPONSORING AGENCY	SUMMARY AND NOTES
EXPECTED DAMAGE AND ANALYSIS		
Harries 2006 **CASE STUDY**	PennDOT	This report focused on discussing the reasons behind the collapse of the Lake View Drive Bridge in PA. An analysis was conducted on bridge members to determine the strength of members (the collapsed girder failed under dead load). From observations, it is suggested that for each corrosion-damaged strand, a conservative measurement of 50% of the surrounding steel is also corrosion-damaged.
Naito et al. 2006 **CASE STUDY**	PennDOT	This report summarizes the findings of a forensic evaluation on beams from the Lakeview Drive Bridge. Included in the report are in-situ material properties, as-built member dimensions, remaining prestress, chloride profile in the web and flanges of the beams, depth of carbonation, concrete quality and corrosion and spalling conditions and its damage.
Russell (2009)	PennDOT	This study's objective is simplifying the analysis of exterior girders so that a simple plane sections approach (as is applied in Section 5) may be used for exterior girders subject to biaxial bending. This study is currently in progress.
FRP REPAIR APPLICATION ISSUES		
Carolin et al. 2005	SBUF, and Swedish Road Authorities	This paper presents laboratory tests investigating the effect of live loads during the strengthening process. For the beams strengthened by laminate plate bonding or NSMR with epoxy, the cyclic loads do not significantly affect the strengthening effect for the tested beams. The failures were controlled by anchorage in the concrete, and the absolute effect on the bond behavior could not be directly studied from the load-bearing capacity.
Setunge et al. 2002	CRC Construction Innovation	Well done as an overview of FRP's: from properties (not specifics but general principles) to ideal uses. Interesting design suggestions are given for safety factors, failure type and equations. More importantly, it is suggested in repair design that creep, relaxation and shrinkage have already occurred meaning that understanding of these phenomena is important so as to accurately judge and assess how the strength and behavior of the member.
OTHER RESOURCES CONSULTED		
AASHTO (2007) AASHO (1960) ACI 318-08 (2008) ACI 440.2R-08 (2008) Broomfield and Tinnea (1992) Cadei et al. (2004) Chadwell and Imbsen (2002)	Collins and Mitchell (1997) Grabb-it (2008) Klaiber et al. (2004) Law Engineering (1990) Mirmiran et al. (2004) Oehlers and Seracino (2004) PaDOH (1960a)	PaDOH (1960b) Sika (2008a) Sika (2008b) Sika (2008c) Spancrete (1960) Williams (2008)

3.2.1 Prestressed CFRP Anchorages

It is apparent from the literature review that although prestressed and post-tensioned CFRP systems behave very well and can affect SERVICE as well as STRENGTH limits states, their anchorage is a challenge. In prestressed CFRP applications, the prestressing force in the CFRP strip must transfer into the girder through the bonding agent (adhesive). Due to the high strains at the bond interface, strip debonding is a major concern. It is essential that the entire force be transferred into the beam via the adhesive layer or the repair will not behave as designed and fail prematurely. Additionally, most suitable high performance epoxy adhesives exhibit significant creep and are therefore unsuitable for maintaining a large prestress force without additional anchorage. If mechanical anchors are left in place, the system is a post-tensioned CFRP system (which can be bonded or unbonded). Permanent anchors can be used to resist the prestressing force and reduce the chance of early debonding and peeling failures (Wight et al. 2001, El-Hacha et al. 2003, Kim et al. 2008a and Yu et al. 2008b). The anchors at the ends of the CFRP strips reduce the shear deformation that occurs within the adhesive layer associated with the prestress force minimizing the possibility of premature failure (El-Hacha et al. 2003). It is noted that the ability of a system to transfer shear, regardless of anchorage or adhesive used, is limited by the shear capacity of the concrete substrate. ACI 440.2R (2008) recommends that the shear stress transferred is limited to 200 psi in any event.

El-Hacha et al. (2003) tested three different metallic anchors including a round bar, elliptical bar and a flat plate anchor. The results indicated that a flat plate anchor was the most efficient anchor and reinforcement of the anchor zone with CFRP U-wrap resulted in greater failure loads. When the CFRP U-wrap was used in conjunction with the anchorage, failure occurred away from the anchor zone. Although these results seem promising, there are concerns about galvanic corrosion of the anchor when steel and CFRP strips are in direct contact. Mitigation of galvanic corrosion is conventionally addressed by providing an insulating layer, often E-glass (Cadei et al. 2004). This layer is softer than the CFRP and therefore affects the efficiency of the stress transfer.

U-wrapped CFRP strips have been employed as an alternative to metallic anchorage systems (Kim et al. 2008a, Kim et al. 2008b and Yu et al. 2008b). Many nonmetallic mechanical anchoring systems for the CFRP U-wraps have been explored including (Kim et al. 2008a and Kim et al. 2008b): a) CFRP U-wrap; b) mechanical anchorage; c) prestressed CFRP U-wrap with mechanical anchorage; and d) CFRP wrap anchored systems (see Figure 3.6). Test results indicated that: a) the beams with nonmetallic anchors exhibited a pseudoductile failure due to the contribution of CFRP anchors, b) beams with mechanically anchored U-wraps and side sheets exhibited a capacity close to that of the control beam; and c) the beams fitted with nonmetallic anchors displayed better stress redistributions compared to the beam with steel anchors (Kim et al. 2008b).

It has been shown that when an anchorage system is used, the anchored prestressed sheets fail at a greater load than the nonanchored prestressed sheets since anchorage greatly reduces the chance of premature 'end peel debonding' failure of the repair (Wight et al. 2001, El-Hacha et al. 2003, Kim et al. 2008a, Kim et al. 2008b and Yu et al. 2008b).

One unique approach did not use anchors, but rather gradually reduced the prestressing force of the strip until the force was zero at the ends of the strip (Aram et al. 2008). The concept behind this was that peeling failure of the strip could be avoided if the force at the strip terminations is zero. Results show that the gradient anchorage method was not effective and premature debonding failure occurred.

The only known commercially available 'standardized' prestressed CFRP system (i.e.: not customized for each application) is made by SIKA Corporation and marketed primarily in Europe. The SIKA CarboStress system is shown in Figure 3.7. 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 given later in Table 5.4. 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 3.7a) or by the jacking hardware (Figures 3.7b and d). One anchor is the fixed or 'dead' end (Figure 3.7a) while the other is the jacking end (Figure 3.7b). 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 3.7c) near the jacking end. Anchor points can also be located at the beam diaphragms. The introduced stress in the strips can vary according to the structural needs 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). Herman (2005) reports an application of this system on two prestressed concrete box girder bridges. The intended repair of the prestressed concrete box girders was to restore flexural capacity as well as replace some of the lost prestressing forces; employment of the CarboStress system as the repair technique proved successful at restoring flexural capacity and prestressing force. Additionally, this method saved monetary and material resources and minimized construction time and traffic closures.

3.3 Aesthetic Repairs

While beyond the expressed scope of the present work, it is informative to briefly review methods for providing aesthetic repairs to prestressed concrete elements. These repairs are typically necessary to address defects that occur during fabrication, shipping, handling or erection of a member. The methods and extensive guidance on their application is 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).

3.3.1 Patching

No matter how much care is exercised in placing and curing concrete, defects occur that require patching. Patching and repair works require 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. Tapered edges of a patch will break or the thinner sections will weather and spall easily. 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 (Shanafelt and Horn 1985; Feldman 1996) in order to slightly 'prestress' the patch to resist 'pop-out'. Clearly, this is not always possible in *in situ* repairs. The preload technique is covered in Section 5.12

There are six main methods for patching. Their selection will depend on the size of the patch and limitations of each method. Limitations include availability of materials and equipment. The six methods are described in the following sections.

3.3.1.1 Drypack Method

The drypack method should be used for the repair of newly placed concrete in which holes have a depth nearly equal to the smallest dimension of the section, such as in the case of core or bolt holes. The drypack method should not be used on shallow surfaces or for filling a hole that extends entirely through the section or member.

In order to apply the drypack, the surface must be wetted to a 'saturated-surface dry' condition. When there is no excess water, a bond coat is applied. The bond coat is made up of cement and fine sand in a 1:1 ratio. This coat is applied evenly to the patch surface. Then, the drypack is prepared by using cement and sand in ratios of 1:1 to 1:3 by volume. Since the material amounts dealt with are relatively small, volume proportioning is used instead of weight. The drypack mix is then placed in layers having a maximum thickness of 0.5 in. Each layer is tamped separately and thoroughly. The finished patch should be well-cured for durability and strength.

3.3.1.2 Mortar Patch Method

This method is used in concrete members with shallow defects, which require a thin layer of patching material. Honeycombs, surface voids or areas where concrete has been pulled away with the formwork require mortar patching.

Surface preparation, mix design and application are very similar to drypack patching. The main difference is, since mortar patching is applied to shallow defects, the patch itself is more susceptible to the effects of shrinkage or poor bond strength especially near its edges. The use of mixes richer than the 1:3 cement to sand ratio exacerbate the effects of shrinkage whereas leaner mixes will result in poor strength.

3.3.1.3 Concrete Replacement Method

This 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 going entirely through the section, or if the defect goes beyond the reinforcement layer, or in general if the volume is large.

All unsound concrete should be chipped off for initial preparation. If there is reinforcement in the area, at least 1 in. of space should be provided around it. Any displaced or cut-off reinforcement should be repaired providing as much overlap as possible when tying in new reinforcement. A bond coat is then applied to the entire patch area. The repair concrete should be placed while this bond coat is still tacky. The initial layer of concrete should have a slump of around 3 in. with each consecutive layer having lower slump. Each layer should be vibrated by an immersion-type vibrator or a hammer from outside of the form if a vibrator is not available. The patch should then be moist cured for as long as possible.

3.3.1.4 Synthetic Patching

There can be some 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.

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, they are hard to finish smoothly and usually result in a color difference between the patch and the base concrete, clearly showing the repaired section, unless precautions are taken against it.

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, it was found that latex was especially useful in situations where feathered edges cannot be avoided.

3.3.1.5 Prepackaged Patching Compounds

There are many commercial patching products available. There are several points to consider when using such products:

- A good patching material should have an initial setting time of 10 – 15 minutes and a final setting time of 20 – 45 minutes. Cements with setting times other than these should be avoided.
- Some products must be used in small amounts because they generate excessive amounts of heat which can lead to shrinkage and poor durability, and therefore are not suitable for general purpose patching.
- A concrete patch should have a compressive strength at least equal to the base concrete. The compressive strength claimed by many products is based on very low water-cement ratios. For practical applications however, the compressive strength was found to be lower than Type 1 Portland cement mortar.

3.3.1.6 Epoxy Injection

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

Before injection, the crack must be properly sealed at its surface in order to accommodate the pressure required to completely fill the void. In most applications, sealing the crack surface with an epoxy paste is adequate. The area adjacent to the crack should be sound, clean, dry and free of dirt or oil. Epoxy injection ports need to be located based on crack length and depth. Although there are no exact values or rules, rules-of-thumb suggest that ports should not be placed closer than 8 in. For cracks narrower than 0.007 in., locate ports on both faces of a crack in order to verify complete injection. For larger cracks, reverse ports are not required.

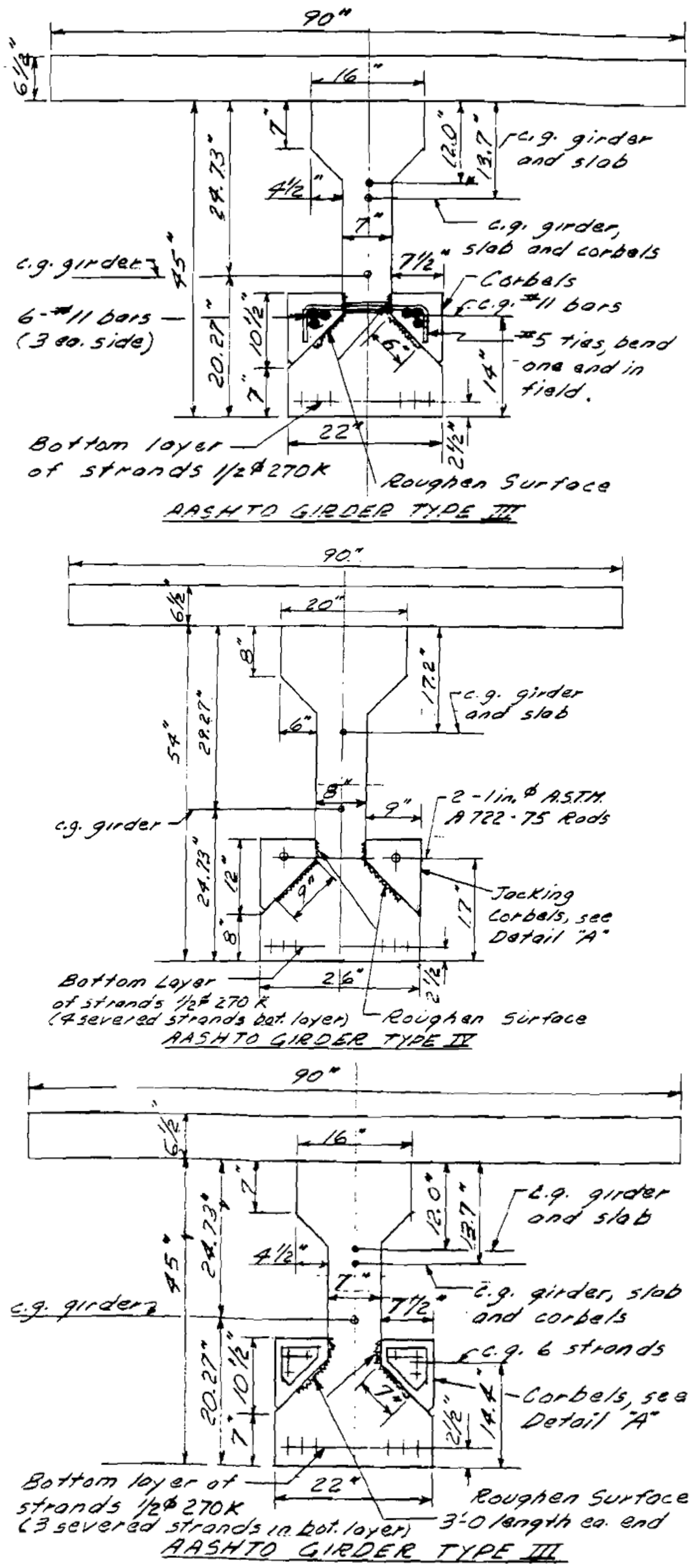
Usually, the maximum pressure the seal or the injection equipment is capable of sustaining is used to reduce the time required to fill the crack and to ensure full penetration. Pressures used in conventional applications range from 40 – 500 psi; most commonly in the range of 75 – 200 psi.
The injection process should continue until either the adjacent port expels the resin or until the pumping motion of the equipment stops. The process should be repeated for each adjacent port in a logical path until refusal is reached at the last port.

Table 3.1 Repair selection criteria (Shanafelt and Horn 1980)

Damage Assessment Factor	Repair Method			
	External PT	**Strand Splicing**	**Steel Jacket**	**Girder Replacement**
Behavior at Ultimate Load	Excellent	Excellent	Excellent	Excellent
Overload	Excellent	Excellent	Excellent	Excellent
Fatigue	Excellent	Limited	Excellent	Excellent
Adding Strength to Non-Damaged Girders	Excellent	N/A	Excellent	N/A
Combining Splice Methods	Excellent	Excellent	Excellent	N/A
Splicing Tendons or Bundled Strands	Limited	N/A	Excellent	Excellent
Number of Strands Spliced	Limited	Limited	Large	Unlimited
Preload Required	Perhaps	Yes	Probably	No
Restore Loss of Concrete	Excellent	Excellent	Excellent	Excellent
Speed of Repair	Good	Excellent	Good	Poor
Durability	Excellent	Excellent	Excellent	Excellent
Cost	Low	Very Low	Low	High
Aesthetics	Fair*	Excellent	Excellent	Excellent

N/A = not applicable

*Can be improved to excellent by extending corbels on fascia girder



(a) Splice 1: mild reinforcing anchored by bolster. PT provided by preload.

(b) Splice 2: PT bar anchored by bolster. Bar is usually mounted in duct. PT provided by jacking or preload.

(c) Splice 4: Prestressing strand in continuous bolsters. Strand may be harped. PT provided by jacking.

Figure 3.1 External post-tensioned repair methods. (Shananfelt and Horn 1980)

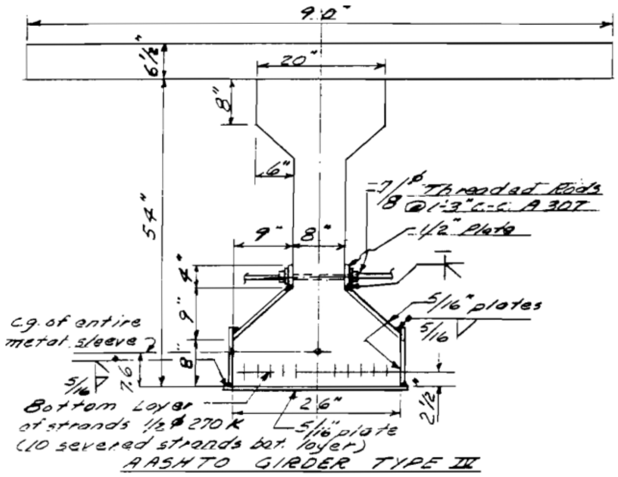


Figure 3.2 Splice 3: Steel jacket. PT provided by preload. Detail shown does not include shear transfer mechanism which is believed to be necessary. (Shanafelt and Horn 1980)

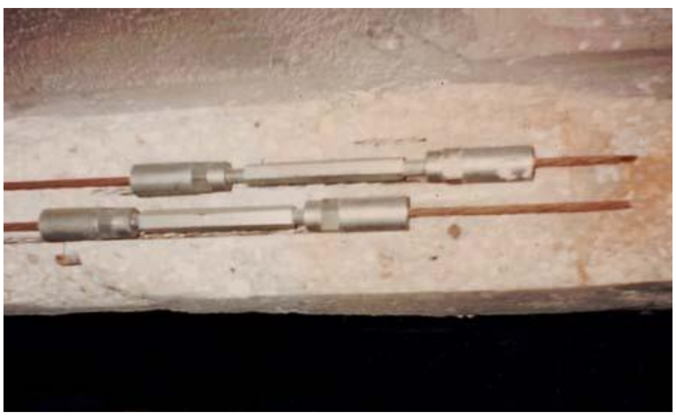
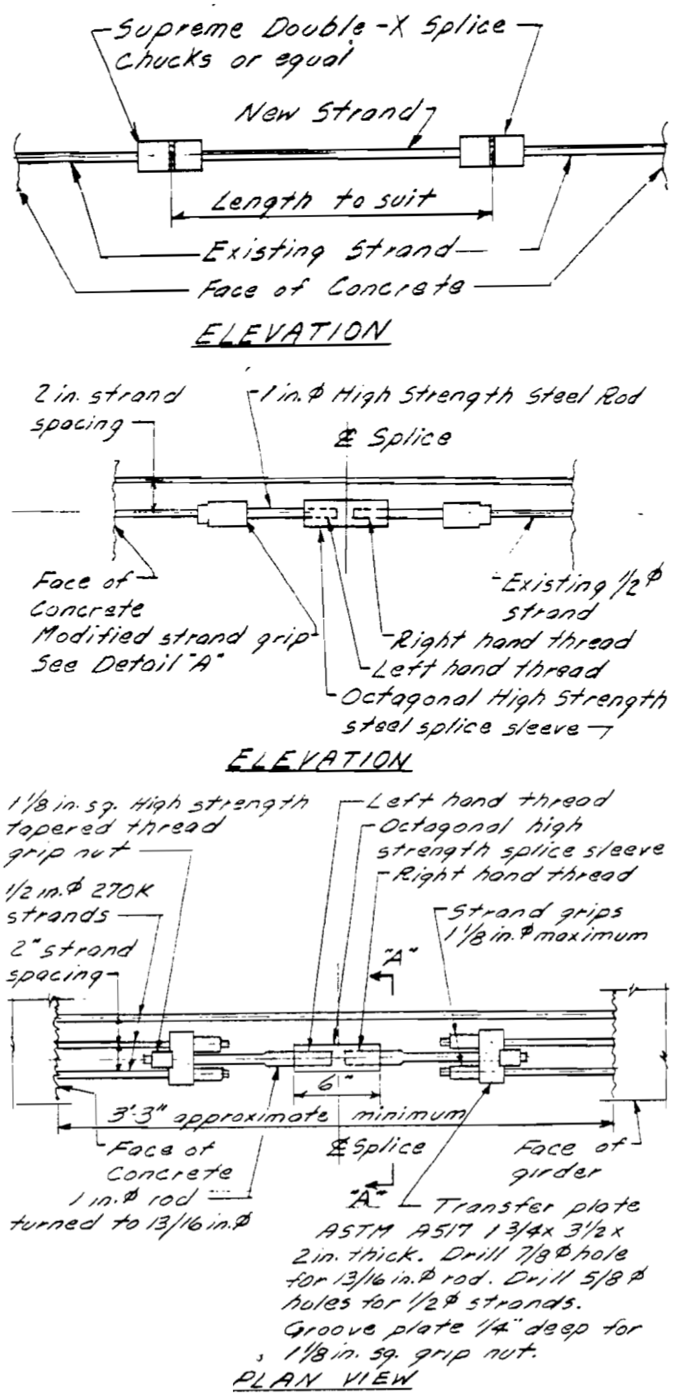


Figure 3.3 Commercially available 'turnbuckle' style strand splice (PCI.org)



(a) Splice 6: Splice chucks used to splice strand. Prestressing reintroduced by heating strand during installation.

(b) Splice 7: ‘Turnbuckle’ style strand splice. Coupler draws strand ends together.

(c) Splice 8: Multiple strand ‘turnbuckle’ style strand splice.

Figure 3.4 Strand splicing methods. (Shananfelt and Horn 1980)

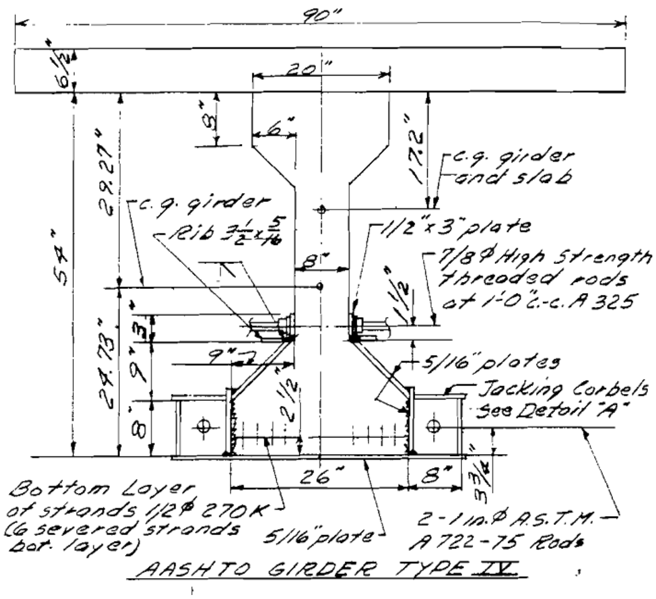


Figure 3.5 Splice 5: Combination of external PT and steel jacket. In this case steel jacket also anchors PT. (Shanafelt and Horn 1980)

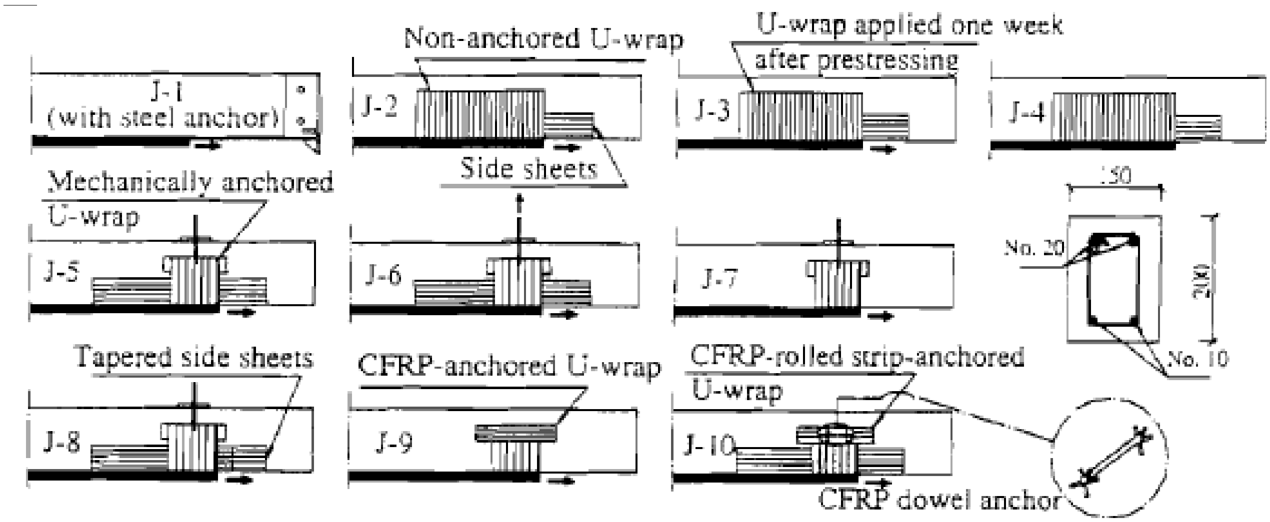
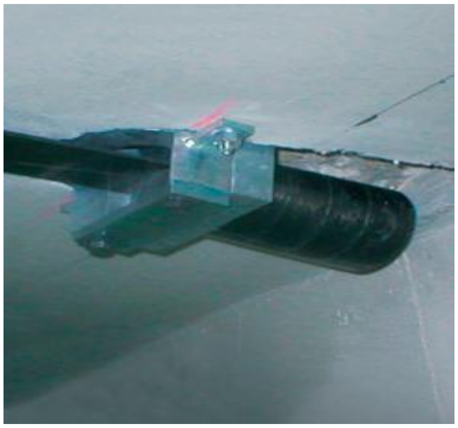
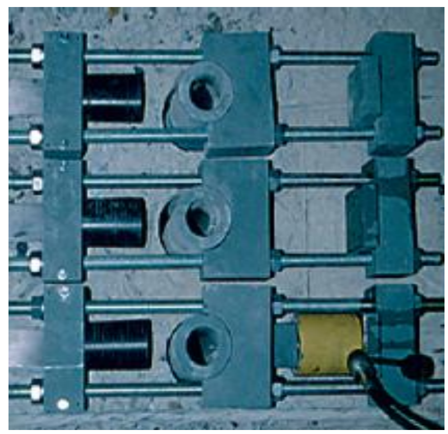


Figure 3.6 Non-metallic anchoring systems (Kim et al. 2008a).



(a) dead end anchor.

(b) jacking end anchor in movable frame.



(c) multiple live end anchors at one location.

(d) stress head system.

Figure 3.7 Sika CarboStress system (SIKA).

TASK 4: SURVEY OF CURRENT STATE OF PRACTICE

A survey of current practice was disseminated to all US State DOTs and all PennDOT Districts through the PennDOT Bureau of Planning and Research. A copy of the survey instrument and accompanying letter is appended to this report. Only fourteen (14) responses were received from eleven (11) DOTs and three (3) PennDOT Districts. The complete responses to the survey follow this section including all notes provided by respondents. Key findings resulting from this limited survey are as follows:

As should be expected, load capacity is the dominant consideration when selecting a repair method. Cost of the repair and interruption of service was also major considerations although duration of the repair operation was not significant, indicating that non-invasive repair techniques or those that limit bridge closure are preferred. The expected durability and/or life extension imparted by the repair is also a consideration indicating that the repair philosophy continues to recognize that our bridge structures will be expected to serve beyond their intended life span.

The overwhelming cause of reported damage is vehicle impact. About 70% of estimated damage is classified as minor damage (defined as: concrete cracks and nicks; shallow spalls and scrapes not affecting tendons). Despite anecdotal evidence to the contrary, few respondents reported corrosion (whether resulting from impact or not) as a significant source of damage. For minor damage, generally no repair action is taken. While 'no action' may seem an initially prudent response for 'minor damage', such damage is unlikely to remain 'minor'. The dominance of minor damage in the survey response suggests an opportunity to deploy preventative measures addressing this level of damage in order to prevent the damage from worsening through ongoing deterioration mechanisms (primarily corrosion).

Moderate damage (defined as: large concrete cracks and spalls; exposed, undamaged tendons) accounted for about 15% of the reported damage and was generally reported to be repaired using surface concrete patching techniques. Significant damage (exposed and damaged tendons; loss of portion of cross section) was rarely reported. In such cases, "load bearing repairs" were generally reported. All existing methods of strand repair offered in Question 11 were reported with similar frequency, indicating no dominate or preferred method of repair. This additionally suggests that, as may be expected, repair methods and designs are site-specific. Severe damage (damage severe enough to result in girder distortion or misalignment), was rarely reported and the consensus was that such damaged girders would be replaced. Washington DOT provided a clear guideline differentiating when repair or replacement would be considered based on the nature of the damage.

In most reported cases, load rating of the damaged girders was conducted using available (often in-house) software. The inspection, on which these models are based, however, is almost exclusively visual. Few jurisdictions report the use of NDE methods. This finding reflects a number of stressors on the bridge inspection process, but significantly the fact that prestressed structures are very difficult to inspect in the first place. The finding additionally suggests the need for more efficient NDE methods for prestressed structures are required.

Although there was little consensus on repair methods or practices, all respondents identified their repairs as successful. One respondent noted that FRP-based repairs were particularly effective, while another had questions regarding the longevity of CFRP systems. Some respondents apparently believe that replacing the girder is the best alternative whereas at least one respondent preferred to monitor the structure for evidence of further deterioration before replacing, due to the high capital cost associated with replacing a girder.

4.1 RESPONDING AGENCIES

Arizona
Florida
Illinois
Kansas
Montana
New Hampshire
New Jersey
South Dakota
Texas
Washington
Wyoming (blank)
Pennsylvania 3-0
Pennsylvania 6-0
Pennsylvania 11-0

4.2 SURVEY RESULTS/RESPONSES

- Boxed numbers indicate the number of respondents indicating the given responses (with the exception of Question #4). Many surveys had multiple responses to a given question.
- Where comments are provided, these are prefaced by the two letter state identifier.
- Comments have not been edited although some typographical errors have been corrected.

1. Plans and specifications for repair of damaged prestressed girders are usually prepared by:

9	DOT/District design/bridge engineer
5	DOT/District maintenance engineer
5	Private consultant
2	Standard details/specifications used
2	Other (please describe)

MT: In the case of beam replacement, the district bridge engineer generates the design and specifications for replacement.

TX: We have a standard specification for repair of damaged girders ftp://ftp.dot.state.tx.us/pub/txdot-info/cmd/cserve/specs/2004/standard/s788.pdf and we include standard plan notes for tendon splicing requirements.

2. Construction of repair is usually carried out by:

11	DOT/District/agency personnel
11	Private contractor
	Other (please describe)

MT: Private Contractor only when replacing a girder.

3. Rate the following factors by importance in the determination of the method or repair.

	low	moderate	high	not considered
Cost of repair	3	8	3	
Time required to make repair		8	6	
Aesthetics of repair	8	6		
Interruption of service		3	10	
Load capacity		1	13	
Expected service life of repair		6	8	
Maintenance required	2	4	6	2
Other, please specify:			1	

IL: Insure concrete patch will be securely anchored & will not eventually loosen & spall off.
KS: shoring or falsework required

4. Estimate the number of prestressed concrete bridges damaged in your jurisdiction over the last five years for the following degrees of damage:

	AR	FL	IL	KS	MT	NH	NJ	SD	TX	WA	PA3	PA6	PA11
Minor	1	200	3	3-6	Lots	5	50	3	250+		15	25	20
Moderate	1	20	3	2-4	12	0	10	1	100	24	15	15	5
Significant		10	3	0-1	6	0	5	0	50	14	4	10	0
Severe		1	2	0-1	2	0	1	1	25	10	0	3	0

5. What actions are typically taken for the following degrees of damage?

	no repair made	non-structural repair	load-carrying repair	replace member or structure
Minor damage	13	2		
Moderate damage	1	11	2	1
Significant damage		1	11	4
Severe damage			2	13

6. What are the typical causes of damage to prestressed concrete members <u>that require repair action to be taken</u>:

common	rare	
13	1	Vehicle impact
2	11	Corrosion occurring at unrepaired site of vehicle impact
3	11	Corrosion resulting from source **other than** vehicle impact
	12	Natural hazard (earthquake, hurricane/tornado, etc.)
1	12	Construction error (misplaced reinforcing steel, low strength concrete, etc.)
1	12	Nonspecific deterioration due to aging
1	1	Other (please describe)

NH: Common: ASR
WA: Rare: Fire Damage
PA11: Corrosion source: Leaking Exp dams

7. In cases where repair action is eventually taken, what procedures are used to determine the extent of the damage?

commonly	rarely	
13		Visual inspection only
1	10	Non destructive evaluation (NDE/NDT); which methods are typically used? **FL:** Load tests used. **NJ:** NDT has not been used in recent past, but would be considered. **SD:** Sounding with hammer
1	9	Destructive evaluation; which destructive methods are typically used? **NJ:** Never used per my knowledge. **WA:** Damaged concrete removed
	1	Other (please describe)

8. Briefly describe what analytical procedures are used to assess the damage and the need for repair of prestressed concrete elements

AR: We have a small amount of concrete prestressed girder overpass structures. We have not had any severely damaged. Most of our bridges are steel I-beam. No analytical procedures at this time to assess the damage and the need for repair.

FL: Various Software packages used, in addition load tests are used.

IL: An initial analysis is performed to decide if the damaged member can be repaired or needs to be replaced. If missing concrete is to be replaced; measures must be taken to insure that the area of concrete repair will remain in place, anchors should be installed in the existing beam and a preload should be applied to the structure during the repair and curing period. The preload used in the analysis should approximate the affect on the member of the passage of a maximum legally loaded vehicle crossing the structure. An analysis is required to determine: 1. The existing stresses in the damaged member. 2. The stresses in the damaged member during the preloading. 3. The stresses in the repaired member after the preloading in removed. If the analysis shows excessive stresses in the member during the necessary repair procedure, the member should be replaced.

KS: load rate vitis/opis or stadd model

MT: A load rating is done using hand calculations, or more recently Virtis, to determine the minimum number of strands needed to obtain/retain an HS-20 capacity.

NJ: A calculation would be performed assessing the loss of prestressing forces which has occurred due to exposed, de-bonded or broken tendons along with the loss of concrete section at the various locations of impact. The load carrying capacity would be assessed to determine what type or repair is necessary or if complete member or superstructure replacement is required.

PA3: Our common procedure for beams that have been damaged is to do a field inspection and obtain measurements for the concrete loss and to verify if strands have been damaged. We will than analysis the beam using PSLRFD or PS3 software program to determine beam capacity with losses.

PA6: The damaged beam is visually inspected to determine extent of strand damage/deterioration. The load rating is revised to account for the lost strand, and a determination is made if any weight limit of traffic restrictions is needed. Software: Prestressed Concrete Girder (3.5)+hand calculations.

PA11: Conventional linear analysis using PennDOT`s PS3 computer program

PR: Damage to prestressed concrete elements is commonly assessed by visual inspection. Members (Beams) are commonly replaced by prestressed or steel beams.

SD: Loss of strand embedment/anchorage and load capacity.

TX: We typically use our in-house prestressed girder design program (PSTRS14) and the new PGSuper to analyze the girder for remaining capacity based on the reduced section properties and taking into account any damaged strands. If the analysis indicates that the girder retains sufficient capacity for ultimate loads, we typically just repair the concrete. (Note, most designs are controlled by service load stresses so a damaged girder often can still meet the ultimate load requirements). If the ultimate load capacity has been reduced to below the minimum level, then strengthening is required or we consider replacing the girder.

WA: If a girder has two or more broken strands we will generally splice the strands without analysis. The following guidelines describe damaged girder conditions which require replacement:

- ***Strand Damage***: More than 25% of prestressing strands are damaged/severed.
  If over 25% of the strands have been severed, replacement is required. Splicing is routinely done to repair severed strands. However, there are practical limits as to the number of couplers that can be installed in the damaged area.
- ***Girder Displacements****:* The bottom flange is displaced from the horizontal position more than ½" per 10' of girder length.
  If the alignment of the girder has been permanently altered by the impact, replacement is required. Examples of non-repairable girder displacement include cracks at the web/flange interface that remain open. Abrupt lateral offsets may indicate that stirrups have yielded. A girder that is permanently offset may not be restorable to its original geometric tolerance by practical and cost-effective means.
- ***Concrete Damage at harping point****:* Concrete damage at harping point resulting in permanent loss of prestress.
  Extreme cracking or major loss of concrete near the harping point may indicate a change in strand geometry and loss in prestress force. Such loss of prestress force in the existing damaged girder cannot be restored by practical and cost-effective means, and requires girder replacement.
- ***Concrete Damage at girder ends:*** Severe concrete damage at girder ends resulting in permanent loss of prestress.
  Extreme cracking or major loss of concrete near the end of a girder may indicate unbonding of strands and loss in prestress force. Such loss of prestress force in the existing damaged girder cannot be restored by practical and cost-effective means, and requires girder replacement.

9. Methods of repair of MINOR damage (concrete cracks and nicks; shallow spalls and scrapes not affecting tendons) used are:

common	rare	
13		Do nothing
3	7	Repaint surface
7	5	Patch concrete
		Other (please describe)

10. Methods of repair of MODERATE damage (large concrete cracks and spalls; exposed, undamaged tendons) used are:

common	rare	
1	10	Do nothing
13	1	Patch concrete
9	4	Epoxy injection
12	1	Concrete removal/surface preparation prior to patching
10	2	Clean tendons
1	10	Installation of active or passive corrosion control measures
	1	Other (please describe)

11. Methods of repair of SIGNIFICANT damage (exposed and damaged tendons; loss of portion of cross section) used are:

common	rare	**Repair of tendons**		yes	no
3	5	Cut tendons flush with damaged section (no tendon repair)			
	9	External post-tensioning			
3	7	Internal splices (describe below)	…is repair re-stressed?	3	
3	7	Metal sleeve slice	…is repair re-stressed?	2	1
1	8	Combination splice (describe below)	…is repair re-stressed?		1
3	7	Externally applied reinforcing material (FRP, etc.) (describe below)			
1	8	Installation of active or passive corrosion control measures			
3		Other (please describe)			

FL: The decision to restress the repair would be made on a case by case basis.

MT: Other, common: To partially restress all exposed strands, we use preloading of the beam (i.e. exposed strands) with heavy trucks or other loads, continuously, during concrete pour until concrete has reached a moderate strength (4 to 6 hours). We use a bagged rapid setting concrete product for our concrete patching material which has a published strength of 3,000 to 5,000 psi in 3 hours, depending on temperature.

Internal Splice: Common, Only if our analysis (load rating) shows that the remaining strands are not adequate

PR: Member Replacement

PA6: Perform load rating, taking into account severed/damaged tendons.

Repair of concrete

11	1	Concrete removal/surface preparation prior to patching
10	2	Patch concrete
9	3	Epoxy injection
		Other (please describe)

4	7	Replace individual girder
1	7	Replace bridge

Please describe methods used (Question 11)

AR: Have not had to repair a concrete prestressed girder with **SIGNIFICANT** damage.

IL: Damaged area of the beam shall be cleaned of all loose and spalled concrete and sealant. Any exposed portions of the strands shall be sandblasted. Remove the existing concrete to sound concrete along the edges of the damaged area to a depth of 1" min. to 1 ½" max. Power driven pins shall be placed at 9" alternate centers along the damaged length of the beam. Where strands are exposed, place 1" x 1" x 18 gauge welded wire fabric in repair area and attach it to the pins or strands with wire ties with 1" minimum clearance between the finished surface of the new concrete and the welded wire mesh. All surfaces of existing concrete and reinforcing strands in the area to be repaired shall be coated with an epoxy-resin primer bonding agent. The repair shall be made using concrete meeting all the requirements for Class PS Concrete for precast prestressed concrete members, except the maximum size of the aggregate shall be ½". Place the lower form on the bottom of the beam and compact by vibrating (or other approved methods) the concrete mix into the voids. After accessible voids have been filled and compacted, the top vertical form shall be raised into position and the remaining voids filled and compacted. The sloping upper surface shall be finished to the configuration of the existing PPC-I Beam Flange. Note: Preload system shall be in place during the repair and curing period.

MT: After performing an onsite assessment and later, an analysis of the beam, and it is determined that the beam is repairable by "in-house" resources, the following basic outline for repair is followed: Saw cut ½" deep around the extent of the crushed/damaged concrete. Remove all loose and damaged concrete down to sound concrete. Repair tendons, if needed. Sandblast repair area. Form up the repair area. Condition concrete to SSD. Preload the beam. Pour/pump repair concrete into form. 4 to 6 hours after completion of concrete pour, remove preload. Remove forms. Clean the surface of the beam along all cracks with a dry brush/rag. Blow beam/cracks clean of dust with clean compressed air. Apply epoxy injection ports to cracks. Seal cracks with epoxy paste. Inject epoxy into beam. 2 to 4 hours later, remove epoxy ports. Minor aesthetic touch up to the beam is done, but generally very little time is spent making the beam look "good". Depending on the extent of damage, an average repair using MDT resources (MDT Maintenance personnel and equipment) will take 100 to 150 man-hours.

NJ: Severed tendons would typically be cut flush with the remaining sound concrete and the concrete patched in-kind if the analysis indicates that sufficient load carrying capacity remains in the damaged member. If adequate load carrying capacity does not remain, then replacement of the member is the typical option. I have never seen a member where the tendons have been repaired in NJ.

SD: Removal of delaminated concrete that is unsound. Splice any cut or damaged tendons. Repair concrete spalls and delaminated areas. Apply load to girder prior to concrete repair if needed. Epoxy injection repair of cracks in girder.

TX: We specify a Grab-it Splice Sleeve to re-connect damaged tendons. The contractor must do a test installation using a load cell to determine the necessary torque to achieve the desired tension in the spliced strand. In many cases, we don't have room to splice all of the damaged tendons so we supplement the strand splices with CFRP applications. We have also used CFRP to provide additional lateral stiffness and strength for girders that get hit repeatedly.

WA: Concrete is chipped back to sound concrete. Strands are spliced with "Grab-it" splices. Grab-its are tightened to partially re-stress the broken strands. Concrete is replaced. Girder may or may not be preloaded prior to concrete placement.

PA3: Our past practice for repairing beams involved the following measures.

1. Epoxy inject cracks
2. Exposed tendons are usually painted
3. Concrete is patched
4. FRP repairs are made to bring the beam strength back to original capacity.

PA6: When "significant" damage is noted, we request maintenance. The maintenance follows PennDOT Publication 408. The loose/spalled concrete is removed, then the surface is prepared and the concrete is patched. The area can also be epoxy injected, if needed.

PA11: We replace the superstructure when it is in poor condition.

12. Methods of repair of SEVERE damage (damage severe enough to result in girder distortion or misalignment) used are:

common	rare	Repair of distortion/misalignment
	9	External post-tensioning
1	7	Jacking and re-use of damaged member
6	4	Jacking and replacement of damaged member
1	7	Provision of new/additional permanent supports (extended corbels, etc.)
		Other (please describe)

common	rare	Repair of tendons			
1	7	Cut tendons flush with damaged section (no tendon repair)			
	9	External post-tensioning			
	8	Internal splices (describe below)	…is repair re-stressed?	yes ☐	no ☐
2	6	Metal sleeve slice	…is repair re-stressed?	yes ☐	no ☐
1	7	Combination splice (describe below)	…is repair re-stressed?	yes ☐	no ☐
1	7	Externally applied reinforcing material (FRP, etc.) (describe below)			
	8	Installation of active or passive corrosion control measures			
		Other (please describe)			

Repair of concrete

6	4	Concrete removal/surface preparation prior to patching
4	5	Patch concrete
4	6	Epoxy injection
	2	Other (please describe)

10	1	Replace individual girder
2	6	Replace bridge

Please describe methods used (Question 12)

AR: Have not had to repair a concrete prestressed girder with **SEVERE** damage.
MT: Once it is determined that the girder is not repairable, the district bridge design engineer does the design for a new girder and a project is let to an independent construction contractor.
NJ: For this type damage, repair of the damaged members is not usually considered due to a question of reliability of the repaired member. We would typically replace a member (or maybe 2 members), if the damage is localized. If several members have been impacted, then we would typically replace the entire superstructure.
PA3: We have not had a beam severely damaged.
PA6: Please see Question 11 answer. Also, in some cases where the damage is severe and threatens the structural integrity of the bridge. We may replace a member, again following the procedures set forth in Pub. 408, we jack the bridge and replace the damaged member.
SD: Same methods described for Question 11 for repair. When replacing an individual girder, diaphragm, partial deck and rail are removed. Replacement girder is put in-place, diaphragm, deck and rail replaced.
WA: Concrete is chipped back to sound concrete. Strands are spliced with “Grab-it” splices. Grab-its are tightened to partially re-stress the broken strands. Concrete is replaced. Girder may or may not be preloaded prior to concrete placement.

13. How well do your prescribed repair methods perform? Please identify problems and/or successes experienced in the repair of damaged prestressed concrete bridge elements in your jurisdiction.

AR: Our non-structural concrete patch repairs for MODERATE damage have performed well. No repair experience with SIGNIFICANT or SEVERE damage.
FL: We have found that FRP repairs are particularly effective. Repairs with girders loaded also works well.
KS: If in question remove and replace girder.
SD: Repairs have been successful to date. Not aware of any situations where we had to go in and fix a repair area again, unless it was damaged from another over-height impact.
MT: So far the only problems have been when the girder gets hit again. The repaired areas generally perform similar to an unrepaired beam, when impacted directly on a repaired area. No evidence of corrosion on strands has been noticed in repaired beams.
NJ: Our repair methods are conservative since we don’t typically attempt to repair badly damaged members using post-tensioning or splicing of tendons. That being said, we have not had any performance problems on the damaged girders that have been repaired.

PA3: All of the repairs that we have made have had no issues at this point in time.

PA6: Given the high amount of money required to undertake large scale rehab projects, we typically assess the deterioration and determine the structural/load capacity of the bridge, if we deem it necessary to be closed or rehabbed, we do not hesitate. If, through our investigation it is found that the problems are minor, we`ll usually monitor the problem to see if it has gotten worse, and weight the bridge, if necessary. Posting the bridge allows us to monitor it more closely (annually, rather than bi-annually) and we can also restrict movement for over-weight vehicles.

PA11: Replacing individual beams or superstructures works best. We are interested in some of the repairs mentioned in this survey (external post tensioning, internal splices, metal sleeve splice). Hopefully, we will get some good repair techniques/standards as part of this survey.

PR: The PRHTA use to replace ONLY prestressed beams with severe damage when the rest of the bridge is in an acceptable condition. When the prestressed bridge has a low rating, the bridge is replaced.

TX: We are very satisfied with our repair procedures and methods. We have had numerous repaired girders get hit again and they performed very well – usually suffering damage in a new location. The only outstanding question we have right now is longevity of the CFRP systems. We have been using them for about 6 years and would like more information on their service life.

WA: We have never had a repaired girder fail, or have corrosion of a girder repair require further repair. Girders with repairs due to impact are often impacted again requiring further repair.

NOTES:

NH: I have worked for NHDOT, Bureau of Bridge Maintenance for about 16 years and oversee state maintenance crews that maintain the state owned 2110 bridges greater than 10' span. During my time here we have only done one major repair to a voided slab structure and that was in the early 90's. NH has a low number of prestressed structures most of our bridges are I-Beam Concrete. We regularly have over height trucks scrape the bottom of prestressed concrete beams, but have not had any significant damage. We have about a dozen prestressed I-beam structures that were built in the late 60's and are beginning to show signs of ASR. This has caused longitudinal cracking and we are monitoring it at this time. I did complete the attached survey for the sections that we have had experience in. Hope it helps!

WY sent back a blank survey.

a copy of the survey instrument and distribution cover letter is provided in Appendix B

TASK 5: REPRESENTATIVE REPAIR SCENARIOS

5.1 Prototype Prestressed Girder Selection

It was initially anticipated that specific bridges would be used as prototype structures for repair, however, based on the inventory review (Task 1) it was decided that prototypes will be prepared having greater damage than has been reported on any of the bridges investigated (Table 1.2). For simplicity, only simply supported, non-composite prototypes are considered. There are few continuous prestressed bridge elements and the nature of repair techniques will not generally be affected by whether the structure is composite or non-composite. Based on Task 1, only three bridge types will be considered: a) Adjacent box beams (AB); b) Multi-box (spread box) beams (SB); and c) I-beams (AASHTO-type beams) (IB). Cross sections of the prototype girders used for the repair designs are shown in Figures 5.1, 5.2 and 5.3, respectively. These prototypes are based on the as-built details of bridges LV, A and K, respectively as reported in Table 1.2 and will be described in greater detail in Sections 5.3 and 5.4.

5.2 Damage Classification

The NCHRP 12-21 study (Shanafelt and Horn 1980 and 1985) established three damage classifications: minor, moderate and severe. These are defined in Section 3.1. Based on the potential for more effective retrofit of more heavily damaged members, a further division of the 'severe' category is proposed as follows:

MINOR	Concrete with shallow spalls, nicks and cracks, scrapes and some efflorescence, rust or water stains. Damage at this level does not affect member capacity. Repairs are for aesthetic or preventative purposes.
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.
SEVERE I	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 (ULS).
SEVERE II	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 (SLS) in addition to the ultimate limit state (ULS).
SEVERE III	Damage is too extensive. Repair is not practical and the element must be replaced.

Damage may be quantified in a variety of ways. Table 5.1 may be viewed as a guide for both selecting a method by which to quantify damage to prestressed members and for quantifying the damage. The entries are tentative at this time and were used to guide this design exercise. Based on the findings of the repair scenarios presented and additional parallel studies values will be proposed. Nonetheless, it is informative to describe the approach to damage quantification.

Defining damage based on the number of strands lost is not felt to be rational in so far as this value does not take into account the contribution of an individual strand to the member capacity. That is; 4 strands missing from a girder having only 16 strands is significant, whereas 4 strands missing from a girder having 72 strands may not require immediate repair. Classification by girder deflection, while likely an excellent indicator of performance, is felt to be impractical to establish in the field. Attention is focused on live load and ultimate capacity replacement.

The Washington State DOT response to question 8 of the survey (see Section 4) has provided limited guidance as to when girder replacement is required. This guidance would correspond to the threshold between SEVERE II and SEVERE III. Replacement is required in cases where:

1. Over 25% of the strands have been severed.
2. The bottom flange is displaced from the horizontal position more than ½" per 10' of girder length.
3. If the alignment of the girder has been permanently altered by the impact.
4. Cracks at the web/flange interface remain open.
5. Abrupt lateral offsets may indicate that stirrups have yielded.
6. Concrete damage at harping point resulting in permanent loss of prestress.
7. Severe concrete damage at girder ends resulting in permanent loss of prestress.

Items 3-7 are additional qualitative considerations for determining SEVERE III level damage.

5.3 Repair Examples Selection

Based on the review of repair methodologies available and the proposed damage classification, a 'flow chart' of appropriate repair methods was established for each type of beam considered, adjacent box (AB), multi-box (SB) and AASHTO girder (IB). These flow charts are shown in Figure 5.4. The resulting matrix of repair examples is shown in Table 5.2. Three variants of non-prestressed CFRP, one variant of prestressed CFRP, one variant of post-tensioned CFRP, one variant of strand splicing and one variant of external steel post-tensioning will be demonstrated in examples presented in Section 5.5 to 5.11.

The viable selections outlined in Figure 5.4 were developed based on some practical considerations of girder and retrofit geometry. For example, due to the large dimension of the splices and the need to stagger splices is felt that strand splicing is only marginally applicable in sections having relatively thin wall or flange dimensions (box girders). Such splices would be more appropriate for prestressed slabs having only a single layer of strands and reasonable cover dimensions.

No example of steel jacketing is provided. This method is felt to be very cumbersome to apply in the field and offers no advantages over the non-corrosive, lighter and easier to apply CFRP systems. An example of a steel jacket design is provided in Shanafelt and Horn (1980).

All repair approaches should also include mitigation of the damage source (water ingress, salt spray, etc.), the adoption of passive (sacrificial anodes) or active (impressed current) corrosion mitigation measures and finally concrete patching (see Section 3.3). These steps are shown in Figure 5.4 but are beyond the scope of the present work.

5.4 Prototype Repair Designs

This chapter describes prototype repair designs which include CFRP repairs, strand splicing and steel post tensioning repairs. CFRP repairs are designed primarily using ACI 440.2R-08 *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures* (ACI 2008) as a guide and are based on strain compatibility of the section. Comparable strand splicing and steel post-tensioning repairs are designed using the previously established guidance provided by the NCHRP 12-21 project (Shanafelt and Horn 1985). The objective of this section is to provide design examples where the repair is intended to restore the section flexural capacity of a damaged prestressed girder. The repair method chosen for each girder type and damage is outlined in Table 5.2.

5.4.1 Materials

Section geometry and material properties of the prototype girders are compiled in Table 5.3. CFRP repair materials and post-tensioning steel material properties are compiled in Tables 5.4 and 5.5, respectively. The material strengths and girder geometries used are based on representative/prototype structures LV, A and K as described in Section 5.1. CFRP material and geometric properties are based on manufacturer's data for Sika CarboDur strips (preformed CFRP strips) (Sika 2008a) and SikaWrap Hex 103C (unidirectional CFRP 'fabric') materials. Data for SikaWrap assumes the use of with Sikadur Hex 300 epoxy (Sika 2008c). Post-tensioning steel material and geometric properties are based on the use of 150 ksi Williams all thread bar (Williams 2008). These properties were used for convenience; the use of Sika or Williams products is not specifically endorsed in this document.

5.4.2 Assumptions and Simplifications

For the analysis and repair of the girders some assumptions and simplifications have been made to allow generalized representative designs to be prepared. It is noted that every structure is different and all designs must consider local conditions and circumstances.

All prototype girders are interior girders. It is understood that impact damage is more likely to occur on the exterior girders, but the inclusion of barrier walls complicates the analysis (Harries 2006), clouding the issues relevant in the present work. The main goal is to provide repair designs and model the repaired girder in order to verify the strength of the repair. Therefore, all girders modeled have been considered to be interior and have not included barrier walls. A parallel study (Russell 2009) has as its objective simplifying the analysis of exterior girders so that a simple plane sections approach (as is applied here) may be used for exterior girders subject to biaxial bending.

The design method of FRP repairs accounts for the initial state of the girder by including the strain distribution present at the time of FRP installation in design calculations. The state of strain at the soffit at this time is assumed to be only the strain due to the dead load of the structure. In field applications, additional loads may be presented which need to be included in the calculation of initial strain conditions. Due to limitations of the plane-sections analysis program *XTRACT* (see following section), it is not possible to correctly account for the initial soffit strain for the CFRP repairs. Therefore, the moment curvature plots created for the CFRP repairs are not representative at load levels below the dead load of the structure (of course, the structure will never be subject to loads below this level).

The damage, modeled by removing strands from the section, was chosen to mimic truck impact damage. Strands are removed from the exterior bottom corner and progress inward (this is discussed later in Section 5.4.4). As a result, the section is no longer symmetric and a rotation of the neutral axis occurs resulting a torsional moment being introduced to the girder. Harries (2006) has shown that the effect of this torsional moment is negligible for interior girders (although it can be significant for exterior girders having composite barrier walls). Additionally, the presence of adjacent girders and the coupling effect of the slab further negate the effects of torsion on interior girders. The analyses presented in this document do not account for girder twist.

5.4.3 *XTRACT* Program

XTRACT is the commercial version of the University of California at Berkeley program *UCFyber* (Chadwell and Imbsen 2002). *XTRACT* is a biaxial nonlinear fiber element sectional analysis program. As it is biaxial (2D in the parlance of this report), it permits the input of any section shape. While *XTRACT* can perform moment-curvature (M-ϕ) and axial load-moment interaction (P-M) analyses about the traditional horizontal (x) and vertical (y) axes. Its "orbit analysis" tool additionally permits a M_{xx}-M_{yy} failure surface to be generated based on specified failure criteria. Only moment-curvature analyses are presented in this work.

XTRACT provides both customizable analysis reports and an interactive mode to view results. A strong graphical component allows the user to see the outcome of their analyses. Finally, all data is easily exported in text format for further processing. *XTRACT* is not able to run 'batch jobs' and thus multiple scenarios (as done for this study) require individual runs and data processing. The ease of use (particularly in editing models) of *XTRACT* however makes up for the necessity of this 'brute force' approach for multiple analyses.

The sections analysis design methodology for FRP repair systems is based on strain compatibility and does not consider beam curvature. In modeling the repair designs for the FRP systems, for convenience the target repair capacity has been determined based on the moment capacity at a selected curvature, $\phi = 0.00015$. Because the objective is to consider ultimate capacity, the maximum capacity of the repaired girder, determined from a fiber section analysis (*XTRACT*), is presented in Table 5.6. The ultimate curvature at which this value is achieved is also reported in Table 5.6. The ultimate curvature in all CFRP analyses presented is determined by CFRP debonding failure. While the ultimate curvature varies considerably, all reported values continue to represent a reasonable degree of ductility (see moment-curvature plots in this chapter, i.e. Figure 5.7).

5.4.4 Girder Damage

It is assumed that the most significant damage is related to truck impact. Thus it is appropriate to remove strands beginning at the exterior web-soffit corner and move inward across the soffit of the girder. Even if truck impact is not the source of damage, removing strands in this manner is rational since it represents a worst-case scenario (Harries 2006).

In the analyses to follow, strands were removed from the lower three layers only. The three-digit identification of each analysis indicates the number of strands ***removed*** from the lower, second and third layers, respectively. Thus, IB 6-2-1 indicates 6 strands removed from the lower layer, 2 from the second and 1 from the third, for a total of 9 strands removed from the I-beam section (Figure 5.3). In all cases the strands were removed from the exterior face and moved inward. An example is shown in Figure 5.5. Table 5.6 lists all cases considered. In Table 5.6, the nominal capacity of the damaged girders is given along with the nominal capacity of the undamaged girder ('target capacity'). The objective of all repairs is to restore the undamaged girder capacity. Figures 5.1 through 5.3 show the girder prototypes and their strand arrangement.

5.4.5 Bridge Loading

Bridge load calculations were completed according to AASHTO LRFD (2007) specifications and are compiled in Tables 5.7 to 5.10 for the various girder types. Loads are calculated based on the HS-25 vehicle. It is suggested that in adjacent box (AB) beam bridges with inadequate or damaged shear keys that a moment distribution factor of g = 0.50 be used (Harries 2006). Table 5.8 shows this case and illustrates the potential difference between the assumed load distribution, where the distribution factor is approximately g = 0.30 (Table 5.7) and *possible in situ* conditions (Table 5.8). Most bridges reviewed in this study were originally built around 1960, therefore the bridges were originally designed for a lower HS-20 loading according to the 1960 AASHO Specifications. The HS-20 and HS-25 loads are shown in Tables 5.7 through 5.10 to contrast the difference between current rating loads and original design loads. Select load levels from these tables are superimposed onto the repaired girder moment-curvature plots presented later.

5.5 Non Prestressed Preform CFRP Strip Repairs

Non-prestressed CFRP strip repairs assume the use of Sika CarboDur strips (Table 5.4). The explanation of the repair design is best seen via example. This example illustrates the necessary steps in designing a CFRP repair as well as provides a brief explanation of each step. All equations, equation numbers and clause references shown in the example are from ACI 440.2R-08 unless noted otherwise. The girder and damage considered for this example repair is the AB 4-0-0 case. Subsequent cases refer to the steps described in this example and identify appropriate modifications. A summary of the parameters, intermediate values obtained during the calculations and results of this repair are shown in Table 5.11. Schematic drawings of the resulting repair are presented in Figure 5.6. Non-prestressed perform CFRP strip repairs have been modeled using *XTRACT* and the moment-curvature plots are shown in Figures 5.7 and 5.8.

5.5.1 Design Example AB 4-0-0

The design example is presented below. A brief description of each step and the associated equations are provided in the left column. The calculations associated with AB 4-0-0 are provided in the right column. All subsequent CFRP designs use the approach presented with some modification as indicated in the sections to follow. A summary of resulting design details and their capacities is given in Table 5.6.

In the following example, the capacity of the damaged AB 4-0-0 is 3160 k-ft (Table 5.6). The objective of the repair is to restore the undamaged nominal moment capacity of the girder: 3387 k-ft (Table 5.6).

Procedure	Calculation
Define objective of repair. For all examples discussed, the objective is to restore the undamaged moment capacity, M_u. Values of M_u and the capacity of the damaged girders are given in Table 5.6.	Restore undamaged moment capacity: $M_n = 3387$ k-ft Capacity of damaged girder without repair: $M_{n\ 4\text{-}0\text{-}0} = 3160$ k-ft

Step 1: Calculate the FRP system design material properties. The repair is of a bridge girder exposed to the elements. Per ACI Table 9.1, a reduction factor, C_E, of 0.85 is suggested. $f_{fu} = C_E f_{fu}^*$ $\varepsilon_{fu} = C_E \varepsilon_{fu}^*$	$f_{fu} = 0.85 \times 406ksi = 345ksi$ $\varepsilon_{fu} = 0.85 \times 0.017in/in = 0.0145in/in$
Step 2: Assemble beam properties. Assemble geometric and material properties for the beam and FRP system. An estimate of the area of FRP (*A_f*) is chosen here. If the section capacity does not meet the demand after the completion of all steps in this procedure, the FRP area is iterated upon.	$E_c = 6800psi$ $A_{cg} = 786in^2$ $h = 42in$ $d_p = 38.91in$ $y_t = 20.59in$ $y_b = 21.41in$ $e = 18.28in$ $I = 204000in^4$ $r = 16.11in$ $A_p = 4.48in^2$ $E_{ps} = 28500000psi$ $\varepsilon_{pe} = 0.0048$ $P_e = 616000lb$ $E_f = 23200000psi$ $A_f = 0.556in^2$ $d_f = 42.0in$ $cgstrands = 3.09in$

Step 3: Determine the state of strain on the beam soffit, at the time of FRP installation. The existing strain on the beam soffit is calculated. It is assumed that the beam is uncracked and the only load applied at the time of FRP installation is dead load. M_{DL} is changed to reflect a different moment applied during CFRP installation. If the beam is cracked, appropriate cracked section properties may be used. However, a cracked prestressed beam may not be a good candidate for repair due to the excessive loss of prestress required to result in cracking. $\varepsilon_{bi} = \frac{-P_e}{E_c A_{cg}}\left(1+\frac{e y_b}{r^2}\right)+\frac{M_{DL} y_b}{E_c I_g}$	$\varepsilon_{bi} = \frac{-616000lb}{6800\,psi \times 768in^2}\left(1+\frac{18.28in \times 21.41in}{(16.11in)^2}\right)$ $+\frac{(1199k-ft \times 12000)\times 21.41in}{6800\,psi \times 204000in^4} = -0.001in/in$
Step 4: Estimate the depth to the neutral axis. Any value can be assumed, but a reasonable initial estimate of c is 0.1h. The value of c is adjusted to affect equilibrium.	$c = 0.1 \times 42in = 4.2in$

Step 5: Determine the design strain of the FRP system. The limiting strain in the FRP system is calculated based on three possible failure modes: FRP debonding (Eq. 10-2), FRP rupture (Eq. 10-16) and FRP strain corresponding to prestressing steel rupture (Eq. 10-17). The strain in the FRP system is limited to the minimum value obtained from (Eq. 10-2), (Eq. 10-16) and (Eq. 10-17). $\varepsilon_{fd} = 0.083\sqrt{\dfrac{f_c^{'}}{nE_f t_f}}$ (10-2) $\varepsilon_{fe} = \dfrac{\varepsilon_{cu}(d_f - c)}{c} - \varepsilon_{bi} \le \varepsilon_{fd}$ (10-16) $\varepsilon_{fe} = \dfrac{(\varepsilon_{pu} - \varepsilon_{pi})(d_f - c)}{(d_p - c)} - \varepsilon_{bi} \le \varepsilon_{fd}$ (10-17) where $\varepsilon_{pi} = \dfrac{P_e}{E_p A_p} + \dfrac{P_e}{E_c A_c}\left(1 + \dfrac{e^2}{r^2}\right)$ (10-18)	$\varepsilon_{fd} = 0.083\sqrt{\dfrac{6800psi}{1 \times 23200000psi \times 0.047in}}$ $\varepsilon_{fd} = 0.0066in/in$ $\varepsilon_{fe} = \dfrac{0.003in/in \times (42.0in - 4.2in)}{4.2in} - (-0.001)$ $\varepsilon_{fe} = 0.0271in/in$ $\varepsilon_{pi} = \dfrac{616000lb}{28500000psi \times 4.48in^2} +$ $\dfrac{616000lb}{4700000psi \times 786in^2}\left(1 + \dfrac{(18.28in)^2}{(16.11in)^2}\right)$ $\varepsilon_{pi} = 0.0052in/in$ $\varepsilon_{fe} = \dfrac{(0.035 - 0.0052)(42.0 - 4.2)}{(38.91 - 4.2)} - (-0.001)$ $\varepsilon_{fe} = 0.0326in/in$ Therefore, the limiting strain in the FRP system is $\varepsilon_{fd} = 0.0066in/in$ and the anticipated mode of failure is FRP debonding.

Step 6: Calculate the strain in the existing prestressing steel. The strain in the prestressing steel can be calculated using Eq. (10-22): $$\varepsilon_{ps} = \varepsilon_{pe} + \frac{P_e}{E_c A_c}\left(1+\frac{e^2}{r^2}\right) + \varepsilon_{pnet} \le 0.035$$ ε_{pnet} is calculated for concrete crushing (Eq. 10-23a) or FRP rupture or debonding (Eq. 10-23b). The value used in Eq. (10-22) is based on the failure mode of the system. $$\varepsilon_{pnet} = 0.003\frac{(d_p - c)}{c} \qquad \text{(10-23a)}$$ $$\varepsilon_{pnet} = (\varepsilon_{fe} + \varepsilon_{bi})\frac{(d_p - c)}{(d_f - c)} \qquad \text{(10-23b)}$$	For concrete crushing: $$\varepsilon_{ps} = 0.0048 in/in + \frac{616000 lb}{4700000 psi \times 786 in^2} \times \left(1+\frac{(18.28 in)^2}{(16.11 in)^2}\right) + 0.0248 in/in \le 0.035$$ $$\varepsilon_{ps} = 0.0300 in/in$$ For FRP rupture or debonding: $$\varepsilon_{ps} = 0.0048 in/in + \frac{616000 lb}{4700000 psi \times 786 in^2} \times \left(1+\frac{(18.28 in)^2}{(16.11 in)^2}\right) + 0.0059 in/in \le 0.035$$ $$\varepsilon_{ps} = 0.0111 in/in$$ Therefore, FRP debonding represents the expected failure mode of the system and ε_{ps} = 0.0111 in/in.

Step 7: Calculate the stress level in the prestressing steel and FRP. The stresses are calculated in the prestressing steel and FRP using Eq. (10-24) and Eq. (10-9), respectively. $f_{ps} = 28500psi \times \varepsilon_{ps}$ $(when \quad \varepsilon_{ps} \leq 0.0076)$ or (10-24) $f_{ps} = 250ksi - \dfrac{0.04}{\varepsilon_{ps} - 0.0064}$ $(when \quad \varepsilon_{ps} > 0.0076)$ $f_{fe} = E_f \times \varepsilon_{fe}$ (10-9)	$f_{ps} = 250ksi - \dfrac{0.04}{(0.0111) - 0.0064} = 241.5ksi$ $f_{fe} = 23200000\,psi \times 0.0066in/in = 152ksi$
Step 8: Calculate the equivalent stress block parameters. From strain compatibility, the strain in the concrete at failure can be calculated as: $\varepsilon_c = (\varepsilon_{fe} + \varepsilon_{bi})\dfrac{c}{(d_f - c)}$ The strain $\varepsilon_c^{'}$ corresponding to $f_c^{'}$ is calculated as: $\varepsilon_c^{'} = \dfrac{1.7 f_c^{'}}{E_c}$ Using ACI 318-08, the equivalent stress block factors can be calculated as: $\beta_1 = \dfrac{4\varepsilon_c^{'} - \varepsilon_c}{6\varepsilon_c^{'} - 2\varepsilon_c}$ $\alpha_1 = \dfrac{3\varepsilon_c^{'}\varepsilon_c - \varepsilon_c^2}{3\beta_1 \varepsilon_c^{'2}}$	$\varepsilon_c = \left(0.0066in/in - 0.0001in/in\right) \times$ $\dfrac{4.2in}{42.0in - 4.2in} = 0.0007in/in$ $\varepsilon_c^{'} = \dfrac{1.7 \times 6800\,psi}{40700000\,psi} = 0.0025in/in$ $\beta_1 = \dfrac{4 \times 0.0025 - 0.0007}{6 \times 0.0025 - 2 \times 0.0007} = 0.685$ $\alpha_1 = \dfrac{3 \times 0.0025 \times 0.0007 - (0.0007)^2}{3 \times 0.685 \times (0.0025)^2} = 0.384$

Step 9: Calculate the internal force resultants. Use Eq. (10-25) $c = \frac{A_p f_{ps} + A_f f_{fe}}{\alpha_1 f_c^{'} \beta_1 b}$ (10-25)	$c = \frac{4.48in^2 \times 241ksi + 0.556in^2 \times 152ksi}{0.384 \times (6800\,psi \div 1000) \times 0.685 \times 48}$ $c = 13.6in$
Step 10: Adjust c until estimate creates equilibrium. The value of c calculated in Step 9 must be equal to the estimate in Step 4. If not, choose another value of c and repeat Steps 5 through 9 with the new c value until equilibrium is achieved.	By iteration, *c = 10 in.*
Step 11: Calculate the flexural strength corresponding to the prestressing steel and FRP components. The flexural strength is calculated using Eq. (10-26). The component of flexural strength contributed by the FRP system includes an additional (empirical) reduction factor, ψ. $M_{np} = A_p f_{ps}\left(d_p - \frac{\beta_1 c}{2}\right)$ $M_{nf} = A_f f_{fe}\left(d_f - \frac{\beta_1 c}{2}\right)$ The nominal capacity of the section is found as: $M_n = M_{np} + \psi M_{nf}$	$M_{np} = 4.48in^2 \times 241ksi \times \left(38.91in - \frac{0.728 \times 10.0in}{2}\right)$ $M_{np} = 38132k - in$ $\psi = 0.85$ $M_{nf} = 0.556in^2 \times 152ksi \times \left(42.0in - \frac{0.728 \times 10.0in}{2}\right)$ $M_{nf} = 3242k - in$ $\psi \times M_{nf} = 2755k - in$ The nominal section capacity is: $M_n = 38132k - in + 2755k - in$ $M_n = 40887k - in$ $M_n = 3407k - ft$

Step 12: Verify that the repair provides sufficient strength as compared to the demand on the structure. The area of CFRP provided, A_f, is adjusted and the procedure repeated until the desired flexural capacity is achieved.	$M_n = 3407k - ft$ $M_u = 3387k - ft$ $M_n > M_u$ Therefore, the repair is sufficient.
Design Summary	A_f=0.556 in^2 Use 6-2 in. wide CFRP strips as shown in Figures 5.2a and 5.6.

The outlined approach is easily programmed as a spreadsheet (as was done for this study) allowing the designer to investigate the effects of varying any of the parameters with relative ease. The iteration procedures (c and A_f) are also easily automated.

Following the flexural design, the shear capacity should be verified. If the flexural capacity is increased beyond the undamaged girder capacity, the shear demand at ultimate capacity will increase. Typically, for long prestressed highway bridge girders, shear will not be a problem provided the objective of the repair is to simply restore the undamaged capacity of the girder.

The use of 2 in. CFRP strip width in the examples is arbitrary. However, Ramanathan and Harries (2008) have shown that, analogous to reinforcing steel, a larger number of less wide strips (i.e.: using 2-2 in. strips instead of 1-4 in. strip) results in marginally improved debonding performance. Based on interaction of adjacent strips it is recommended that the clear spacing between strips be greater than 0.25 in. (Oehlers and Seracino 2004). Finally, where possible, the strips should be located in the vicinity of the damaged strands. For example, the repair of AB 4-0-0 would likely be arranged as shown in Figure 5.9.

A summary of all non prestressed CFRP strip repairs (AB 4-0-0, AB 8-2-1, SB 4-0-0 and SB 8-2-1) is provided in Table 5.11. Resulting CFRP repairs are shown in Figure 5.6. Finally, detailed moment-curvature responses of: a) the undamaged beams (target values); b) damaged beams; and c) repaired beams are shown in Figures 5.7 and 5.8 for the AB and SB examples, respectively. Also shown in these figures are the 1960 AASHO and 2007 AASHTO design moment and dead load moments for the girders (Tables 5.7 through 5.10).

A fiber section analysis (*XTRACT)* is used to determine the moment-curvature response of each beam. Modeling the repairs using a fiber sections analysis is more refined since the material stress strain behaviors are better captured than in a simplified plane section analysis utilizing stress block factors. Therefore, the results of the sections analysis of Step 11 and the *XTRACT* program are slightly different.

The moment-curvature plots produced to model the repairs (such as Figures 5.7 and 5.8) display a pronounced ‘kink’ in the curves representing section cracking. This kink is an artifact of the transition from uncracked to cracked behavior and is typical of the moment curvature response of prestressed concrete elements as shown in Figure 5.10 (Collins and Mitchell 1997).

5.5.2 Further Examples

The following sections report other repair methods utilizing the preceding detailed example. The sections highlight the differences in parameters and equations used in this method. Like the presented AB 4-0-0 example, each section includes summary tables of the procedure followed, summary drawings of the resulting designs and moment-curvature plots of the target and repaired beam behaviors.

5.6 Non Prestressed CFRP Fabric Repair

The difference between this and the previous repair is the CFRP material. The CFRP fabric is flexible and can be wrapped around complex shapes and thus is particularly useful for 'wrapping' the complex tension flange shape of an I-beam. However, the fabric should not be wrapped around the entire bulb since 'pull off' failures at inside corners can occur easily. Additionally, a significant amount of effort is required to wrap over a sharp corner because the corner must be rounded to accommodate the CFRP fabric. Typically, fabric manufactures recommend a minimum outside corner radius of 1 in. and do not recommend wrapping around an inside corner (such as the flange-to-web interface in an I-beam). Therefore, repairs conducted with the fabric are practically restrained to the bulb only (consisting of the bottom soffit and the vertical sides). The repairs conducted for the IB 6-2-1 and IB 10-2-1 cases use multiple layers of fabric on the soffit (as seen in Figure 5.11). With the exception of CFRP material properties (Table 5.4), the repair design is identical to that presented in Section 5.5.1. Input parameters and results are shown in Table 5.12 and drawings of the repairs are shown in Figure 5.11. The repairs are modeled in *XTRACT* and moment-curvature plots are shown in Figure 5.12. It is noted that the repairs prescribed for IB 6-2-1 and 10-2-1 did not completely restore the undamaged girder moment capacity. This will be discussed in Section 6.

5.7 NSM CFRP Repairs

The design of near-surface mounted (NSM) CFRP repairs is similar to that for CFRP strips presented in Section 5.5. The geometric difference is that the CFRP of an NSM repair is located in the concrete cover of the member (see Figure in Nordin and Taljsten 2006, Section 3.2) thereby affecting the FRP lever arm, d_f, in Step 11. The same material is used for NSM repair as the CFRP strip repair, although the geometry of the material is customized by cutting the strips longitudinally. For the repairs done here, a strip size of 0.875 in. x 0.047 in. was used (see following section for rationale). Additionally, two strips were glued together and inserted into each slot in the beam. This method of increasing the available area of CFRP per slot has been successfully demonstrated by Aidoo et al. (2006), among others. The advantage of an NSM repair is that a greater debonding strain can be achieved. The design of an NSM repair is the same as the example in Section 5.2.1 with the exception of the calculation of equation (10-2) in Step 5. For NSM, rather than making the calculation of equation (10-2), the debonding strain is calculated by $\varepsilon_{fd} = k_m \times \varepsilon^*_{fu}$, (where $k_m = 0.7$) (ACI 440.2R-08). Input parameters and results are shown in Table 5.13 and drawings of the repairs are shown in Figure 5.13 NSM repaired girder moment-curvature plots are seen in Figure 5.14. It is noted that the repair prescribed for IB 10-2-1 did not completely restore the undamaged girder moment capacity. This will be discussed in Section 6.

5.7.1 NSM Strip Size Optimization

NSM slot geometry (required slot size and spacing) is prescribed by ACI 440.2R-08. Therefore, for a given soffit width, an optimal strip size can be determined so as to maximize the area of NSM reinforcement that may be provided. A typical slot, cut with a concrete saw is 0.25 in. wide (Aidoo 2004 and

Quattlebaum et al. 2005). This is the maximum width for the cut (if made in one pass) and therefore restricts the width of NSM reinforcement that may be used[4]. ACI 440.2R-08 recommends that the slot be at least 3 times the width of the inserted strip. Based on this, it is assumed that two strips (glued together) may be inserted into a 0.25 in. slot; this was demonstrated by both Aidoo (2004) and Quattlebaum et al. (2005). The clear concrete cover depth also restricts the NSM strip size. The depth of the slot must not exceed the clear cover as this will result in cutting into the transverse reinforcement. Some margin is required when cutting slots. For prestressed construction where dimensions are well controlled and primary reinforcement does not sag, a margin of 0.125 in. is suggested. Therefore, for the I-beam, for instance, the maximum depth of cut was determined using the depth to the strand (2 in.) and subtracting half of the diameter of the strand (0.219 in.), the diameter of #3 stirrups (0.375 in.) and the safety margin (0.125 in.). Therefore, the maximum slot depth was determined to be approximately 1.25 in. Finally, slot spacing and edge distance is a function of slot depth; ACI 440.2R-08 recommends that spacing exceed twice the slot depth and edge distance be four times the slot depth. Considering these restrictions, an optimal slot size may be determined such that the amount of CFRP is maximized for a given soffit dimension. The optimized NSM reinforcement size for the 24 in. soffit of the IB chosen for NSM repairs is 0.875 x 0.094 in. Allowing for the slot to be 0.125 in. deeper than the CFRP dimension, this arrangement requires 1 in. deep slots located 2 in. on center having a 4 in. edge distance. The optimization process is summarized in Table 5.14.

5.8 Prestressed CFRP Strip Repair

CFRP strip dimension and material properties are based on Sika CarboDur strips. This system does not use mechanical anchorage; therefore the prestressing force is transferred to the beam over the entire bond length of the strip. Since no anchorage is used, it is suggested that CFRP U-wraps be used to help mitigate the possibility of peeling failure at strip ends (Klaiber et al. 2003, Green et al. 2004, Reed and Peterman 2004, Reed and Peterman 2005, Scheibel et al. 2001, Tumialan et al. 2001, and Wipf et al. 2004). Experiments have shown that a sustained prestress force of 30% of the ultimate strain capacity of the strip is achievable (El-Hacha et al. 2003) with a prestressed CFRP system; this value is used in the present example. The differences in design of the prestressed CFRP strip repair as compared to the example presented in Section 5.5.1 are as follows:

1. The strain introduced by the prestressed strip is considered in the calculation of the initial soffit condition, ε_{bi}: (Step 3)

$$\varepsilon_{bi} = \frac{-\left(P_e + 0.30 f_{fu} A_f\right)}{E_c A_{cg}}\left(1 + \frac{e y_b}{r^2}\right) + \frac{M_{DL} y_b}{E_c I_g}$$

2. Adding the anchored strain of the prestressed strip to the debonding strain, ε_{fd} : (Step 5, Equation 10-2)

$$\varepsilon_{fd} = 0.083\sqrt{\frac{f_c^{'}}{n E_f t_f}} + 0.30\varepsilon_{fu}$$

The prestressed CFRP repair design follows the same procedure as the example with the exception of the changes noted in steps 3 and 5, respectively. Input parameters and results are shown in Table 5.15

[4] Alternate methods of cutting the slot include using a concrete grinding wheel (very inefficient), tuck pointing blade (rather inefficient for concrete) or making multiple, overlapping passes with a concrete saw (efficient, but each pass doubles the cost of the slot). Each of these approaches would allow a wider slot to be formed.

and drawings of the repairs are shown in Figures 5.15 to 5.17. Prestressed CFRP repaired girder moment-curvature plots are seen in Figures 5.18 to 5.20. It is noted that the repair prescribed for IB 10-2-1 did not completely restore the undamaged girder moment capacity. This will be discussed in Section 6.

5.9 Bonded Post-Tensioned CFRP Repair

Bonded post-tensioned CFRP repairs include the use of mechanical anchorage at each end of the beam. As a result, a greater strain can be sustained when compared to the prestressed CFRP system described in the previous section. Sika CarboStress system technical data suggests that 50% of the CFRP strip's ultimate strain can be sustained. This value is used in present example. CFRP anchorage is discussed below. Design of bonded post-tensioned CFRP repairs is the same as that of the prestressed CFRP repair design except that the debonding strain, ε_{fd} , calculated in Step 5, is increased to 50% of the strip's ultimate strain (rather than 30% described in the previous section). Additionally, the original state of strain in the soffit, ε_{bi} (Step 3) is also calculated accounting for the amount of post tensioning provided the CFRP. Since this system includes anchorage at the ends, peeling failures are not a concern. Input parameters and results are shown in Table 5.16 and drawings of the repairs are shown in Figures 5.21 to 5.23. Post-tensioned CFRP repaired girder moment-curvature plots are seen in Figures 5.24 to 5.26.

5.9.1 Anchorage of CFRP

CFRP anchorage is usually secured to proprietary anchorage hardware which in turn is anchored to the concrete substrate. The CFRP-to-anchor connections may rely on adhesive bond, friction or bearing of a preformed CFRP 'stresshead' (the SIKA system uses the latter as shown in Figure 3.6a; Sika 2008b). Manufacturer recommendations must be followed in considering the CFRP to-anchor connection.

The proprietary anchor, in turn, is secured to the concrete substrate. Anchor bolts (Figure 3.6c) and shear keys are conventional methods of transferring the force. 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, anchorages will have to be staggered longitudinally (analogous to staggering reinforcing steel lap splice locations) if a large amount of CFRP is required. Temporary jacking anchorages may be bolted or utilize temporary shear keys. An example of a temporary shear key comprised of a xx-strong pipe inserted into a hole cored through the beam web is shown in Figure 3.6d.

For anchorages bolted to the concrete substrate, the recommendations ACI 318-08 Appendix D for bolting to concrete should be followed. 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 resist any moment and to keep the shear key fully engaged. In cases where the end of the beam is available for anchorage (see Figure in Wight et al. 2001 in Section 3.2), this is preferred although bearing stresses should be considered in designing the prestressing anchorage.

5.10 Strand Splice Repair

Conceptually, the goal of a strand splice is to recreate the original strand, including the prestressing force. Due to geometric constraints of concrete cover, strand spacing and strand splice dimensions, this repair can only be used to repair a small number of strands at a particular section. The 'turn of the nut method' is suggested (rather than the torque wrench method) to ensure that the proper stress is reintroduced in the strand (Labia et al. 1996 and Olson et al. 1992). Determining the amount of stress introduced into the strand by the strand splice is done using the stiffness of the strand splice and the stiffness of the undeveloped strand (i.e.: at least the exposed strand being connected) and balancing these with the 'shortening' of the splice as the nut is turned. The stiffness of the strand splice is a function of its geometry, length and strand diameter being developed. This stiffness must be calculated on an individual basis. Based on the desired prestress force, P, stiffness of the strand splice, K_{splice}, exposed length of strand, $L_{exposed}$ and strand transfer length, L_{tr} into the concrete, the required shortening of the strand splice may be calculated as:

$$\Delta_{splice} = \frac{P}{K_{splice}} + \frac{P\left(\sum L_{\exp osed} + L_{tr}\right)}{A_p E_p} \qquad \text{(Eq. 5.1)}$$

A schematic of the lengths discussed in Equation 5.1 are presented in Figure 5.27.

For the I-beam, for instance, the stress in the 7/16 in. strand after long term losses was found to be 133.6 ksi. Suggested practice is to add 5 ksi for dead load stress and 5 ksi for error to the target stress value and use this value as the target value for the strand splice induced stress (Labia et al. 1996). This resulted in a target stress of 143.6 ksi (corresponding to a force of 15.5 kips) per strand. Assuming a splice stiffness of 187.7 k/in. (reported by Labia et al. 1996), that there is 24 inches of exposed strand to either side of the splice and that the strand transfer length is equal to $d_b(f_{pe}/3000) = 21\ in.$ (ACI 318-08), a shortening of 0.42 in. is required. There are 16 threads per inch on the splice (Grabb-it 2008); therefore, to reach the required deformation, 6.7 nut revolutions are required. The use of the strand transfer length assumes a linear development of strand force in the sound concrete. Thus the strand strain associated with development of the strand force is $PL_{tr}/2A_pE_p$. Considering both sides of the splice, the ½ coefficient cancels and Equation 5.1 results.

The use of the preload technique is often used with the strand splice method. The preload technique is discussed in Section 5.12.

5.11 External Steel Post-Tensioning

The goal of external steel post-tensioning is to restore the compressive stress in the bottom of the girder as intended by the original prestressed strands as well as increase the flexural capacity. Although not covered in this document, external steel post tensioning can be used to restore original stress levels in the bottom of the girder even if there is no damage. In this document, this method is used to repair the IB 6-2-1 and 10-2-1 cases.

Analysis of the section after strand loss is done by sections analysis. A general procedure is provided here as an example.

1. Determine the amount of stress lost at the girder soffit due to the loss of strands

   $$f_{loss} = \left(-\frac{P}{A} - \frac{Pe}{S} + \frac{M_{DL}}{S}\right)_{undamaged} - \left(-\frac{P}{A} - \frac{Pe}{S} + \frac{M_{DL}}{S}\right)_{damaged} \qquad \text{(Eq. 5.2)}$$

   It should be noted that the section modulus, S, and effective area, A, may be different for the undamaged and damaged terms particularly if the damaged girder is cracked under the influence of dead load. The P and Pe terms are the axial prestressing force and its resulting moment (e is the strand eccentricity), respectively. The M_{DL} term is the moment due to girder dead load.

2. Determine the required force in the post tensioning steel needed to replace the lost strands:

$$f_{loss} = \left(-\frac{P}{A} - \frac{Pe}{S}\right)_{PT} \qquad \text{(Eq. 5.3)}$$

3. Design the bolster for the post-tensioning system. The bolster should anchor the additional forces and should be designed such that in the event of overstress, the post-tensioning bar, rather than the bolster, fails.

5.11.1 Design Example IB 6-2-1

The design example is presented below. A brief description of each step and the associated equations are provided in the left column. The calculations associated with IB 6-2-1 are provided in the right column. In the following example, the capacity of the damaged IB 6-2-1 is 3731 k-ft (Table 5.6). The objective of the repair is to restore the undamaged nominal moment capacity of the girder: 4590 k-ft (Table 5.6).

Procedure	Calculation
Define objective of repair. For all examples discussed, the objective is to restore the undamaged moment capacity, M_u. Values of M_u and the capacity of the damaged girders are given in Table 5.6.	Restore undamaged moment capacity: $M_n = 4590$ *k-ft* Capacity of damaged girder without repair: $M_{n\ 6\text{-}2\text{-}1} = 3731$ *k-ft*
Step 1: Assemble beam properties. Assemble geometric and material properties for the beam and external steel post-tensioning system. An estimate of the location of the post-tensioning steel is chosen here (given as e_{PT}).	$P_D = 591.6k$ $P_{UD} = 721.4k$ $A = 1272 in^2$ $S = 12212 in^3$ $e_D = 26.1 in$ $e_{UD} = 26.8 in$ $e_{PT} = 11.0 in$ $M_{DL} = 1372 k - ft$

Step 2: Verify that the damaged girder properties. The girder will have different properties from the original design when it is cracked. Therefore, verify that the girder remains uncracked. The girder will remain uncrakced if the equation below holds true. $$\frac{M_{DL}}{S} < \left(\frac{P}{A} + \frac{Pe}{S}\right)_{damaged}$$	$$\frac{1372 \times 12}{12212} < \left(\frac{591.6}{1272} + \frac{591.6 \times 26.1}{12212}\right)_{damaged}$$ $$1.35ksi < (0.47 + 1.26)ksi$$ $$1.35ksi < 2.53ksi$$ Therefore, the section remains uncracked.
Step 3: Calculate the lost stress at the girder soffit using Equation 5.2. $$f_{loss} = \left(-\frac{P}{A} - \frac{Pe}{S} + \frac{M_{DL}}{S}\right)_{undamaged} - \left(-\frac{P}{A} - \frac{Pe}{S} + \frac{M_{DL}}{S}\right)_{damaged}$$	$$f_{loss} = \left(-\frac{721.4}{1272} - \frac{724.1 \times 26.8}{12212} + \frac{1372 \times 12}{12212}\right)_{undamaged} - \left(-\frac{591.6}{1272} - \frac{591.6 \times 26.1}{12212} + \frac{1372 \times 12}{12212}\right)_{damaged}$$ $$f_{loss} = (-0.8)_{undamaged} - (-0.38)_{damaged}$$ $$f_{loss} = -0.42ksi$$
Step 4: Determine the required force in the post-tensioning steel using Equation 5.3. $$f_{loss} = \left(-\frac{P}{A} - \frac{Pe}{S}\right)_{PT}$$	$$-0.42ksi = \left(-\frac{P}{1272} - \frac{P \times 11.0}{12212}\right)_{PT}$$ $$P = 248k$$
Step 5: Select the appropriate post-tension rod size per manufacturer specification. Rod size is selected based on the permissible force in the rod.	Choose 1.25” diameter 150 ksi rod (Williams 2008).
Design Summary	Use 2, 1.25” diameter 150 ksi rods tensioned to 99 ksi and anchored 20” above the girder soffit (e_{PT}=11.0 in).

Drawings of the example repairs are shown in Figures 5.28 and 5.29 and the repaired girder moment-curvature plots are seen in Figure 5.30.

Post-tensioning steel will typically take the form of pairs of solid high strength post-tensioning rods (such as Williams all-thread bars) or prestressing strand. Due to the dimension of the post-tensioning system and the possibility of impact damage, external post-tensioning systems are conventionally mounted along the girder web rather than the soffit below. As a result, this repair method is inappropriate for adjacent box girders. Appropriate environmental protection (such as using encapsulated strand, epoxy-coated or galvanized rod, etc.) is provided for external applications.

Bolsters can be made of either concrete or steel. Bolster material is the preference of the designer, but cost and constructability must be considered. Regardless of bolster material, bolster design is to be carried out as a shear friction connection following AASHTO (2007) Section 5.8.4. Figure 5.31a shows an example of a concrete bolster and Figure 5.31b shows a schematic of a steel angle bolster.

5.12 Preload Technique

Preload is the application of a load to a girder during the repair process. Used primarily to improve the performance on concrete patches, the preload results in a tension stress applied to the beam soffit. The patch is executed in this condition and when the preload is released, the patch is drawn into compression (even if there is still a net tension at the soffit). The goal of a preload is to sufficiently compress the concrete patch in order to counteract live load effects reducing the possibility of patch 'pop-out' failure. Although covered in this document for completeness, it should be realized that this method is not applicable for all structures or repair types.

A generalized preload application procedure is provided here as an example (adopted and corrected from Labia et al. 1996).

1. Using AASHTO (2007) Table 5.9.4.2.2-1, the maximum permissible tensile stress, f_t, at the bottom of the patch can be selected. Typically a value of $0.19\sqrt{f_c^{'}}$ (ksi units) is selected.
2. The maximum external moment, M_{EXTmax}, that can be applied can be determined as follows:
$$f_t \leq -\frac{P}{A}\left(1+\frac{ey_b}{r^2}\right)+\frac{M_D}{S_d}+\frac{M_{EXT\max}}{S_d}. \quad \text{(Eq. 5.4)}$$
3. For completeness, compressive stress due to the prestressing force and dead load at the bottom of the damaged girder should be checked using Table 5.9.4.2.1-1 (AASHTO 2007). These stresses should not exceed $0.45 f_c^{'}$:
$$-\frac{P}{A}\left(1+\frac{ey_b}{r^2}\right)+\frac{M_D}{S_d} \leq 0.45 f_c^{'}. \quad \text{(Eq. 5.5)}$$

Upon release of the preload, the concrete patch is placed in compression with a stress equal to M_{EXT}/S_d. Due to the magnitude of the load required to achieve a useful value of M_{EXT}, the use of preloading is only practical on shorter spans.

Table 5.1 Proposed damage classifications.

Damage Classification	**SEVERE I**	**SEVERE II**	**SEVERE III**
Repair philosophy	ULS only	ULS and SLS	-
Action	non PT repair	PT repair	replace
Live load capacity replacement	up to 5%	up to 30%	100%
Ultimate load capacity replacement	up to 8%	up to 15%	100%
Replace lost strands	2-3 strands	up to 8 strands	>8 strands
Deflection	loss of camber	up to 0.5%	>0.5%

Table 5.2 Repair Examples.

Beam	**Damage**	**Retrofit**
Adjacent Box Beam	4-0-0 & 8-2-1	Non-prestressed preformed CFRP strip
	8-2-1	Prestressed CFRP strips
	8-2-1	Post-tensioned CFRP strips
Spread Box Beam	4-0-0 & 8-2-1	Non-prestressed preformed CFRP strip
	8-2-1	Prestressed CFRP strips
	8-2-1	Post-tensioned CFRP strips
AASHTO I-girder	4-0-0	Strand Splice
	4-0-0, 6-2-1 & 10-2-1	Non-prestressed CFRP fabric
	4-0-0, 6-2-1 & 10-2-1	Non-prestressed NSM CFRP
	4-0-0, 6-2-1 & 10-2-1	Prestressed CFRP strips
	4-0-0, 6-2-1 & 10-2-1	Post-tensioned CFRP strips
	6-2-1 & 10-2-1	External steel post-tensioning

Table 5.3 Prototype girder material and geometric properties.

Property	**AB**	**SB**	**IB**
Section	prestressed concrete adjacent box beam	prestressed concrete multi-box beam	prestressed concrete I-girder
prestressing steel	60 - 250 ksi 3/8 in. seven-wire strand	68 - 250 ksi 3/8 in. seven-wire strand	50 - 250 ksi 7/16 in. seven-wire strand
Young's modulus of prestressed steel, E_p	28500 ksi	28500 ksi	28500 ksi
Concrete girder compressive strength, f_c'	6800 psi	5500 psi	5500 psi
Young's modulus of girder, E_c	4700 ksi	4227 ksi	4227 ksi
Concrete deck compressive strength	n.a.	4000 psi	4000 psi
Young's modulus of deck	n.a.	3605 ksi	3605 ksi
girder geometry	Figure 5.1	Figure 5.2	Figure 5.3
girder length	90.0 ft	69.0 ft	75.5 ft

Table 5.4 CFRP material and geometric properties (Sika 2008a and 2008c)

Property	**Sika CarboDur strips**	**SikaWrap Hex 103C (w/Sikadur Hex 300 epoxy)**
Material type	preformed unidirectional CFRP strip	unidirectional CFRP fabric
Tensile strength, f_{fu}	406 ksi	104 ksi
Compressive strength	-	-
Young's Modulus, E_f	23,200 ksi	9,446 ksi
Rupture strain, ε_{fu}	0.017	0.0098
Material thickness	0.047 in.	approx. 0.04 in.
Size/packaging	1.97 in. strips[1] 3.15 in. strips 3.94 in. strips	25 in. x 50 ft. rolls 25 in. x 300 ft. rolls
[1] product is fabricated in 50, 75 and 100 mm widths; hard conversions are presented here to facilitate later stress calculations.		

Table 5.5 Post-tensioning steel material and geometric properties (Williams 2008).

Nominal Bar Diameter	Minimum Net Area Through Threads	Minimum Tensile Strength	Minimum Yield Strength
1.25 in.	1.25 in^2	188 kips	150 kips
1.375 in.	1.58in^2	237 kips	190 kips

Table 5.6 Summary of repair details and capacities.

undamaged girder strand details	Damage	REPAIR Type		Capacity (k-ft)				ε_{fd}
		Detail	A_f (in^2)	damaged girder	undamaged girder (target capacity)	repaired girder	repaired curvature φ	
Adjacent box (AB) girder 60 – 3/8 in. 250 ksi strands	4-0-0	6 – 2” CFRP strips	0.56	3160	3387	3425	0.00019	0.0066
	8-2-1	17 – 2” CFRP strips	1.57	2770	3387	3396	0.00019	0.0066
	8-2-1	8 – 2” P-CFRP	0.74	2770	3387	3590	0.00025	0.0109
	8-2-1	6 – 2” PT-CFRP	0.56	2770	3387	3369	0.00018	0.0138
Spread box (SB) girder 68 – 3/8 in. 250 ksi strands	4-0-0	6 – 2” CFRP strips	0.56	4317	4596	4591	0.00015	0.0059
	8-2-1	18 – 2” CFRP strips	1.67	3838	4596	4822	0.00015	0.0059
	8-2-1	9 – 2” P-CFRP	0.83	3838	4596	4553	0.00013	0.0102
	8-2-1	6 – 2” PT-CFRP	0.56	3838	4596	4461	0.00013	0.0131
AASHTO I (IB) girder 50 – 7/16 in. 250 ksi strands	4-0-0	CFRP fabric	0.80	4200	4590	4596	0.00022	0.0100
	6-2-1	CFRP fabric	3.44	3731	4590	4436	0.00013	0.0058
	10-2-1	CFRP fabric	3.44	3340	4590	4052	0.00013	0.0058
	4-0-0	8 – 7/8” CFRP NSM	0.33	4200	4590	4703	0.00026	0.0119
	6-2-1	22 – 7/8” CFRP NSM	0.91	3731	4590	4972	0.00026	0.0119
	10-2-1	22 – 7/8” CFRP NSM	0.91	3340	4590	4389	0.00026	0.0119
	4-0-0	2 – 2” P-CFRP	0.19	4200	4590	4345	0.00013	0.0102
	6-2-1	9 – 2” P-CFRP	0.83	3731	4590	4492	0.00013	0.0102
	10-2-1	11 – 2” P-CFRP	1.02	3340	4590	4280	0.00013	0.0102
	4-0-0	3 – 2” PT-CFRP	0.28	4200	4590	4502	0.00013	0.0131
	6-2-1	8 – 2” PT-CFRP	0.74	3731	4590	4600	0.00013	0.0131
	10-2-1	12 – 2” PT-CFRP	1.11	3340	4590	4554	0.00013	0.0131
	4-0-0	Strand Splice		4200	4590	4590	0.00015	n.a.
	6-2-1	2 – 1 1/4” dia. 150 ksi rods, tensioned to 99 ksi		3731	4590	4291	0.0001	n.a.
	10-2-1	2 – 1 3/8” dia. 150 ksi rods, tensioned to 103 ksi		3340	4590	4040	0.0001	n.a.

CFRP strips = Non-prestressed preformed CFRP strip
P-CFRP = Prestressed CFRP strips
PT-CFRP = Post-tensioned CFRP strips

Table 5.7 AB loading with AASHTO-prescribed distribution factor g = 0.285.

	based on load…	Moment	MPF	g	IM	Strength I	Service I	Service III	units
M_{DW}	0.12 klf	118	-	-	-	177	118	118	k-ft
M_{SW}	0.90 klf	909	-	-	-	1137	909	909	k-ft
M_{JB}	0.17 klf	171	-	-	-	214	171	171	k-ft
M_{LANE}	0.64 klf	648	1	0.285	-	323	185	148	k-ft
M_{HS20}	HS20	1344	1	0.285	1.33	891	509	407	k-ft
M_{HS25}	HS25	1680	1	0.285	1.33	1114	637	509	k-ft
M_{TAN}	TANDEM	1076	1	0.285	1.33	713	407	326	k-ft
					Dead Load Moment (M_{DL})=	1528	1199	1199	k-ft
					Live Load Moment (HS20) =	1214	694	555	k-ft
					Live Load Moment (HS25) =	1437	821	657	k-ft
					Live Load Moment (TANDEM) =	1036	592	474	k-ft

MPF = multiple lane presence factor
g = distribution factor for moment
IM = impact factor

Table 5.8 AB loading with distribution factor g = 0.5.

	based on load…	Moment	MPF	g	IM	Strength I	Service I	Service III	units
M_{DW}	0.12 klf	118	-	-	-	177	118	118	k-ft
M_{SW}	0.90 klf	909	-	-	-	1137	909	909	k-ft
M_{JB}	0.17 klf	171	-	-	-	214	171	171	k-ft
M_{LANE}	0.64 klf	648	1	0.5	-	567	324	259	k-ft
M_{HS20}	HS20	1344	1	0.5	1.33	1564	894	715	k-ft
M_{HS25}	HS25	1680	1	0.5	1.33	1955	1117	894	k-ft
M_{TAN}	TANDEM	1076	1	0.5	1.33	1252	715	572	k-ft
					Dead Load Moment =	1528	1199	1199	k-ft
					Live Load Moment (HS20) =	2131	1218	974	k-ft
					Live Load Moment (HS25) =	2522	1441	1153	k-ft
					Live Load Moment (TANDEM) =	1819	1039	831	k-ft

MPF = multiple lane presence factor
g = distribution factor for moment
IM = impact factor

Table 5.9 SB loading.

	based on load…	Moment	MPF	g	IM	Strength I	Service I	Service III	units
M_{DECK}	0.77 klf	456	-	-	-	570	456	456	k-ft
M_{DW}	0.20 klf	122	-	-	-	182	122	122	k-ft
M_{SW}	0.80 klf	475	-	-	-	594	475	475	k-ft
M_{JB}	0.09 klf	53	-	-	-	66	53	53	k-ft
M_{LANE}	0.64 klf	381	1	0.648	-	432	247	197	k-ft
M_{HS20}	HS20	968	1	0.648	1.33	1460	834	667	k-ft
M_{HS25}	HS25	1210	1	0.648	1.33	1825	1043	834	k-ft
M_{TAN}	TANDEM	813	1	0.648	1.33	1227	701	561	k-ft
					Dead Load Moment =	1411	1105	1105	k-ft
					Live Load Moment (HS20) =	1892	1081	865	k-ft
					Live Load Moment (HS25) =	2257	1289	1032	k-ft
					Live Load Moment (TANDEM) =	1659	948	758	k-ft

MPF = multiple lane presence factor
g = distribution factor for moment
IM = impact factor

Table 5.10 IB loading.

	based on load…	Moment	MPF	g	IM	Strength I	Service I	Service III	units
M_{DECK}	0.70 klf	499	-	-	-	623	499	499	k-ft
M_{SW}	0.69 klf	491	-	-	-	614	491	491	k-ft
M_{JB}	0.15 klf	108	-	-	-	135	108	108	k-ft
M_{LANE}	0.64 klf	456	1	0.592	-	472	270	216	k-ft
M_{HS20}	HS20	867	1	0.592	1.33	1194	682	546	k-ft
M_{HS25}	HS25	1084	1	0.592	1.33	1493	853	682	k-ft
M_{TAN}	TANDEM	894	1	0.592	1.33	1232	704	563	k-ft
					Dead Load Moment =	1372	1098	1098	k-ft
					Live Load Moment (HS20) =	1667	952	762	k-ft
					Live Load Moment (HS25) =	1965	1123	898	k-ft
					Live Load Moment (TANDEM) =	1705	974	779	k-ft

MPF = multiple lane presence factor
g = distribution factor for moment
IM = impact factor

Table 5.11 Non prestressed perform CFRP repair results.

Step #		AB 4-0-0	AB 8-2-1	SB 4-0-0	SB 8-2-1	units
1	f_{fu}	345	345	345	345	ksi
1	ε_{fu}	0.0145	0.0145	0.0145	0.0145	in/in
2	cg strands	3.09	3.16	4.41	4.77	in.
2	d_f	42	42	50	50	in.
2	d_p	38.91	38.84	45.59	45.23	in.
2	ε_{cu}	0.003	0.003	0.003	0.003	in/in
2	P_e	616	539	692	616	kips
2	A_p	4.48	3.92	5.12	4.56	in^2
2	E_{ps}	28500	28500	28500	28500	ksi
2	A_{cg}	786	786	1553	1553	in^2
2	E_c	4700	4700	4230	4230	ksi
2	e	18.32	18.31	27.44	27.14	in
2	I	204000	204000	543000	543000	in^4
2	r	16.1	16.1	18.7	18.7	in
2	ε_{pe}	0.0048	0.0048	0.0047	0.0047	in/in
2	A_f	0.56	1.57	0.56	1.67	in^2
2	$f_c'{}_{DECK}$	-	-	4000	4000	psi
3	ε_{bi}	-0.0001	0	-0.0002	-0.0001	in/in
4	c	9.9	10	7.5	7.5	in.
5	ε_{fd}	0.0066	0.0066	0.0059	0.0059	in/in
5	ε_{fe} (cc)	0.0098	0.0097	0.0172	0.0172	in/in
5	ε_{pi}	0.0052	0.0052	0.0051	0.0050	in/in
5	ε_{fe} (psr)	0.0331	0.0332	0.0336	0.0339	in/in
6	ε_{pnet} (cc)	0.0088	0.0087	0.0152	0.0151	in/in
6	ε_{pnet} (frp)	0.0058	0.0059	0.0051	0.0051	in/in
6	ε_{ps} (cc)	0.0140	0.0138	0.0203	0.0201	in/in
6	ε_{ps} (frp)	0.0110	0.0110	0.0102	0.0101	in/in
7	f_{ps}	241	241	239	239	ksi
7	f_{fe}	152	152	137	137	ksi
8	ε_c	0.0020	0.0020	0.0010	0.0010	in/in
8	ε'_c	0.0025	0.0025	0.0016	0.0016	in/in
8	β1	0.728	0.730	0.711	0.711	-
8	α	0.811	0.820	0.697	0.701	-
9/10	**c** (check)	10.0	10.1	7.6	7.6	in
11	M_{np}	38132	33253	52593	46388	k-in
11	M_{nf}	3242	9175	3596	10782	k-in
11	ψ_f	0.85	0.85	0.85	0.85	-
11	M_n	40888	41052	55650	55553	k-in
11	M_n	3407	3421	4638	4629	k-ft
12	M_u (Table 5-4)	3395	3395	4596	4596	k-ft

Table 5.12 CFRP fabric repair results.

Step #		IB 4-0-0	IB 6-2-1	IB 10-2-1	units
1	f_{fu}	88.4	88.4	88.4	ksi
1	ε_{fu}	0.0102	0.0102	0.0102	in/in
2	cg strands	6.43	6.78	7.3	in.
2	d_f	52.5	52.0	52.0	in.
2	d_p	46.07	45.72	45.2	in.
2	ε_{cu}	0.003	0.003	0.003	in/in
2	P_e	664	592	534	kips
2	A_p	4.97	4.43	4.00	in^2
2	E_{ps}	28500	28500	28500	ksi
2	A_{cg}	1272	1272	1272	in^2
2	E_c	4230	4230	4230	ksi
2	e	26.45	26.1	25.72	in
2	I	402400	402400	402400	in^4
2	r	17.8	17.8	17.8	in
2	ε_{pe}	0.0047	0.0047	0.0047	in/in
2	A_f	0.8	3.44	3.44	in^2
2	$f_c'{}_{DECK}$	4000	4000	4000	psi
3	ε_{bi}	-0.0002	-0.0002	-0.0001	in/in
4	c	6.3	7.6	6.7	in.
5	ε_{fd}	0.0100	0.0058	0.0058	in/in
5	ε_{fe} (cc)	0.0222	0.0177	0.0204	in/in
5	ε_{pi}	0.0051	0.0050	0.0050	in/in
5	ε_{fe} (psr)	0.0350	0.0350	0.0354	in/in
6	ε_{pnet} (cc)	0.0189	0.0150	0.0172	in/in
6	ε_{pnet} (frp)	0.0084	0.0048	0.0048	in/in
6	ε_{ps} (cc)	0.0240	0.0201	0.0222	in/in
6	ε_{ps} (frp)	0.0135	0.0099	0.0098	in/in
7	f_{ps}	244	238	238	ksi
7	f_{fe}	95	55	55	ksi
8	ε_c	0.0013	0.0010	0.0010	in/in
8	ε'_c	0.0016	0.0016	0.0016	in/in
8	β1	0.731	0.708	0.702	-
8	α	0.822	0.677	0.614	-
9/10	c (check)	6.4	7.7	6.8	in
11	M_{np}	53100	45394	40413	k-in
11	M_{nf}	3798	9247	9241	k-in
11	ψ_f	0.85	0.85	0.85	-
11	M_n	56328	53254	48268	k-in
11	M_n	4694	4438	4022	k-ft
12	M_u (Table 5-4)	4688	4688	4688	k-ft

Table 5.13 NSM CFRP repair results.

Step #		IB 4-0-0	IB 6-2-1	IB 10-2-1	units
1	f_{fu}	345	345	345	ksi
1	ε_{fu}	0.0145	0.0145	0.0145	in/in
2	cg strands	6.43	6.78	7.3	in.
2	d_f	51.9	51.4	51.4	in.
2	d_p	46.07	45.72	45.2	in.
2	ε_{cu}	0.003	0.003	0.003	in/in
2	P_e	664	592	534	kips
2	A_p	4.97	4.43	4.0	in^2
2	E_{ps}	28500	28500	28500	ksi
2	A_{cg}	1272	1272	1272	in^2
2	E_c	4230	4230	4230	ksi
2	e	26.45	26.1	25.72	in
2	I	402400	402400	402400	in^4
2	r	17.8	17.8	17.8	in
2	ε_{pe}	0.0047	0.0047	0.0047	in/in
2	A_f	0.33	0.91	0.91	in^2
2	$f_c'{}_{DECK}$	4000	4000	4000	psi
3	ε_{bi}	-0.0002	-0.0002	-0.0001	in/in
4	c	6.0	6.0	5.7	in.
5	ε_{fd}	0.0119	0.0119	0.0119	in/in
5	ε_{fe} (cc)	0.0232	0.0228	0.0242	in/in
5	ε_{pi}	0.0051	0.0050	0.0050	in/in
5	ε_{fe} (psr)	0.0345	0.0344	0.0348	in/in
6	ε_{pnet} (cc)	0.0200	0.0199	0.0208	in/in
6	ε_{pnet} (frp)	0.0102	0.0103	0.0102	in/in
6	ε_{ps} (cc)	0.0251	0.0249	0.0258	in/in
6	ε_{ps} (frp)	0.0153	0.0153	0.0152	in/in
7	f_{ps}	246	246	245	ksi
7	f_{fe}	276	276	276	ksi
8	ε_c	0.0015	0.0016	0.0015	in/in
8	ε'_c	0.0016	0.0016	0.0016	in/in
8	β1	0.744	0.746	0.740	-
8	α	0.873	0.878	0.859	-
9/10	c (check)	6.0	6.1	5.8	in
11	M_{np}	53464	47240	42242	k-in
11	M_{nf}	4511	12270	12304	k-in
11	ψ_f	0.85	0.85	0.85	-
11	M_n	57298	57670	52701	k-in
11	M_n	4775	4806	4392	k-ft
12	M_u (Table 5-4)	4742	4742	4742	k-ft

Table 5.14 NSM size optimization.

FRP strip width (in)	Depth of slot required (in)	Edge distance required (in)	Required spacing between slots (in)	Number of slots in 24 in. wide soffit	Available area of FRP[1] (in^2)
b_b	$b_b + 0.125$	$4(b_b + 0.125)$	$2(b_b + 0.125)$		
0.500	0.625	2.5	1.25	13	0.306 - 0.611
0.625	0.750	3.0	1.5	11	0.323 - 0.646
0.750	0.875	3.5	1.75	9	0.317 - 0.635
0.875	1.000	4.0	2	8	0.329 - 0.658
1.000	1.125	4.5	2.25	6	0.282 - 0.564
1.125	1.250	5.0	2.5	6	0.317 - 0.635

[1]A range is provided to show the area of FRP using one or two strips per slot, respectively. Actual area of FRP can be anywhere between these bounds.

Table 5.15 Prestressed CFRP repair results.

Step #		AB 8-2-1	SB 8-2-1	IB 4-0-0	IB 6-2-1	IB 10-2-1	units
1	f_{fu}	345	345	345	345	345	ksi
1	ε_{fu}	0.0145	0.0145	0.0145	0.0145	0.0145	in/in
2	cg strands	3.16	4.77	6.43	6.78	7.3	in.
2	d_f	42	50	52	52	52	in.
2	d_p	38.84	45.23	46.07	45.72	45.2	in.
2	ε_{cu}	0.003	0.003	0.003	0.003	0.003	in/in
2	P_e	539	616	664	592	534	kips
2	A_p	3.92	4.56	4.97	4.43	4.00	in^2
2	E_{ps}	28500	28500	28500	28500	28500	ksi
2	A_{cg}	786	1553	1272	1272	1272	in^2
2	E_c	4700	4230	4230	4230	4230	ksi
2	e	18.31	27.14	26.45	26.1	25.72	in
2	I	204000	543000	402400	402400	402400	in^4
2	r	16.1	18.7	17.8	17.8	17.8	in
2	ε_{pe}	0.0048	0.0047	0.0047	0.0047	0.0047	in/in
2	A_f	0.74	0.83	0.19	0.83	1.02	in^2
2	$f_c'{}_{DECK}$	-	4000	4000	4000	4000	psi
3	ε_{bi}	-0.0001	-0.0002	-0.0002	-0.0002	-0.001	in/in
4	c	7.3	6.1	6.2	6.2	6.0	in.
5	ε_{PT}	0.004	0.004	0.004	0.004	0.004	in/in
5	ε_{fd}	0.0109	0.0102	0.0102	0.0102	0.0102	in/in
5	ε_{fe} (cc)	0.0144	0.0218	0.0226	0.0226	0.0234	in/in
5	ε_{pi}	0.0052	0.0050	0.0051	0.0050	0.0050	in/in
5	ε_{fe} (psr)	0.0329	0.0338	0.0349	0.0353	0.0358	in/in
6	ε_{pnet} (cc)	0.0130	0.0192	0.0193	0.0191	0.0196	in/in
6	ε_{pnet} (frp)	0.0098	0.0090	0.0086	0.0086	0.0085	in/in
6	ε_{ps} (cc)	0.0181	0.0243	0.0244	0.0242	0.0246	in/in
6	ε_{ps} (frp)	0.0150	0.0140	0.0137	0.0136	0.0135	in/in
7	f_{ps}	245	245	245	244	244	ksi
7	f_{fe}	253	237	237	237	237	ksi
8	ε_c	0.0023	0.0014	0.0013	0.0013	0.0013	in/in
8	ε'_c	0.0025	0.0016	0.0016	0.0016	0.0016	in/in
8	$\beta 1$	0.741	0.735	0.731	0.731	0.728	-
8	α	0.863	0.840	0.825	0.825	0.811	-
9/10	c (check)	7.4	6.1	6.2	6.3	6.1	in
11	M_{np}	34703	47970	53209	46990	41953	k-in
11	M_{nf}	7348	9449	2209	9931	12156	k-in
11	ψ_f	0.85	0.85	0.85	0.85	0.85	-
11	M_n	40949	56002	55087	55431	52285	k-in
11	M_n	3412	4667	4591	4619	4357	k-ft
12	M_u (Table 5-4)	3388	4596	4557	4557	4557	k-ft

Table 5.16 Post-tensioned CFRP repair results.

Step #		AB 8-2-1	SB 8-2-1	IB 4-0-0	IB 6-2-1	IB 10-2-1	units
1	f_{fu}	345	345	345	345	345	ksi
1	ε_{fu}	0.0145	0.0145	0.0145	0.0145	0.0145	in/in
2	cg strands	3.16	4.77	6.43	6.78	7.3	in.
2	d_f	42	50	52	52	52	in.
2	d_p	38.84	45.23	46.07	45.72	45.2	in.
2	ε_{cu}	0.003	0.003	0.003	0.003	0.003	in/in
2	P_e	539	616	664	592	534	kips
2	A_p	3.92	4.56	4.97	4.43	4.00	in^2
2	E_{ps}	28500	28500	28500	28500	28500	ksi
2	A_{cg}	786	1553	1272	1272	1272	in^2
2	E_c	4700	4230	4230	4230	4230	ksi
2	e	18.31	27.14	26.45	26.1	25.72	in
2	I	204000	543000	402400	402400	402400	in^4
2	r	16.1	18.7	17.8	17.8	17.8	in
2	ε_{pe}	0.0048	0.0047	0.0047	0.0047	0.0047	in/in
2	A_f	0.56	0.56	0.28	0.74	1.11	in^2
2	$f_c'_{DECK}$	-	4000	4000	4000	4000	psi
3	ε_{bi}	-0.0001	-0.0002	-0.0002	-0.0002	-0.002	in/in
4	c	6.3	5.5	5.8	5.8	5.8	in.
5	ε_{PT}	0.007	0.007	0.007	0.007	0.007	in/in
5	ε_{fd}	0.0138	0.0131	0.0131	0.0131	0.0131	in/in
5	ε_{fe} (cc)	0.0171	0.0245	0.0244	0.0244	0.0244	in/in
5	ε_{pi}	0.0052	0.0050	0.0051	0.0050	0.0050	in/in
5	ε_{fe} (psr)	0.0329	0.0337	0.0349	0.0350	0.0358	in/in
6	ε_{pnet} (cc)	0.0155	0.0217	0.0208	0.0208	0.0204	in/in
6	ε_{pnet} (frp)	0.0125	0.0116	0.0162	0.0162	0.0159	in/in
6	ε_{ps} (cc)	0.0207	0.0267	0.0259	0.0259	0.0254	in/in
6	ε_{ps} (frp)	0.0176	0.0166	0.0162	0.0162	0.0159	in/in
7	f_{ps}	246	246	246	246	246	ksi
7	f_{fe}	320	304	304	304	304	ksi
8	ε_c	0.0024	0.0016	0.0016	0.0016	0.0016	in/in
8	ε'_c	0.0025	0.0016	0.0016	0.0016	0.0016	in/in
8	β1	0.748	0.749	0.750	0.750	0.750	-
8	α	0.883	0.887	0.888	0.888	0.888	-
9/10	c (check)	6.5	5.6	5.8	5.9	5.9	in
11	M_{np}	35181	48407	53612	47768	42222	k-in
11	M_{nf}	7033	8105	4256	11345	17015	k-in
11	ψ_f	0.85	0.85	0.85	0.85	0.85	-
11	M_n	41159	55296	57229	57411	56685	k-in
11	M_n	3430	4608	4769	4784	4724	k-ft
12	M_u (Table 5-4)	3388	4596	4742	4742	4742	k-ft

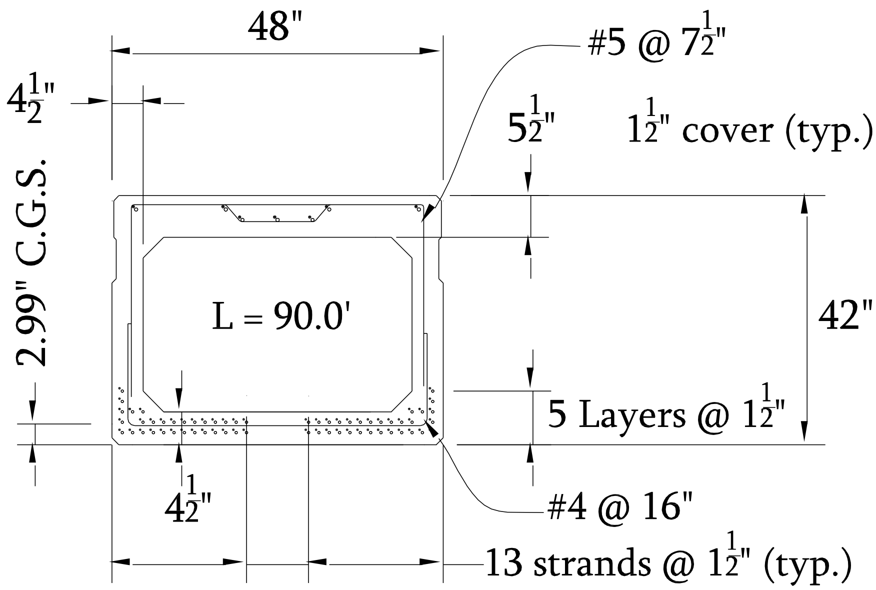


Figure 5.1 Prototype AB girder cross section.

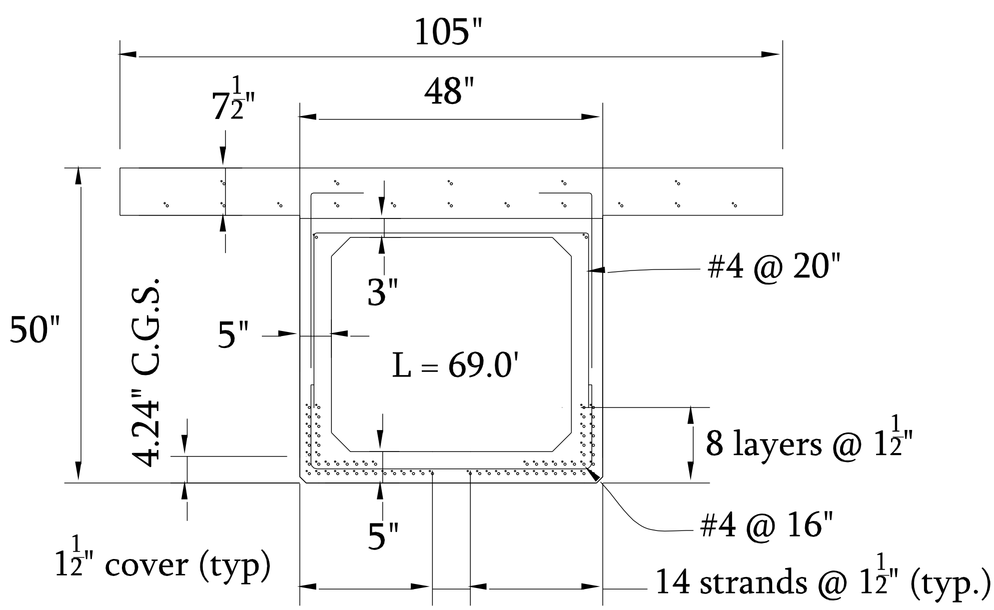


Figure 5.2 Prototype SB girder cross section.

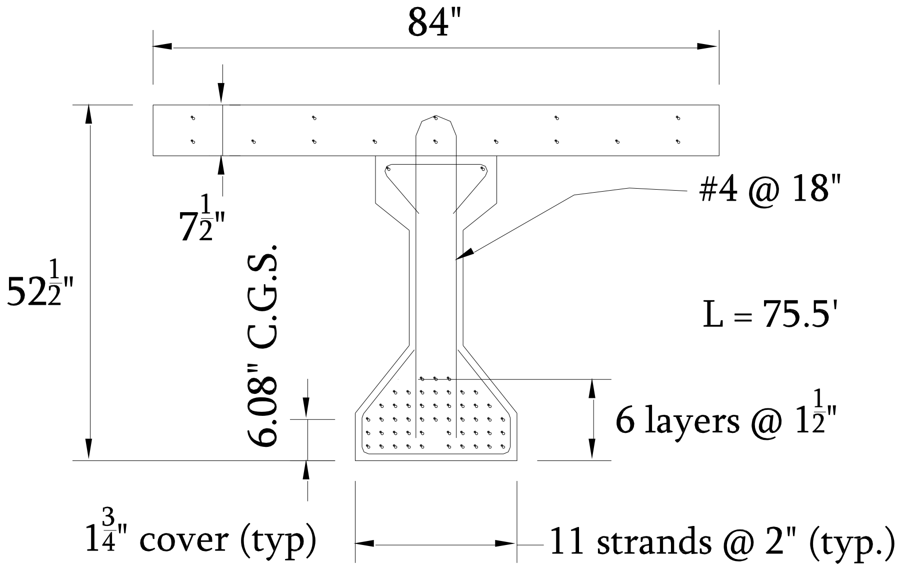
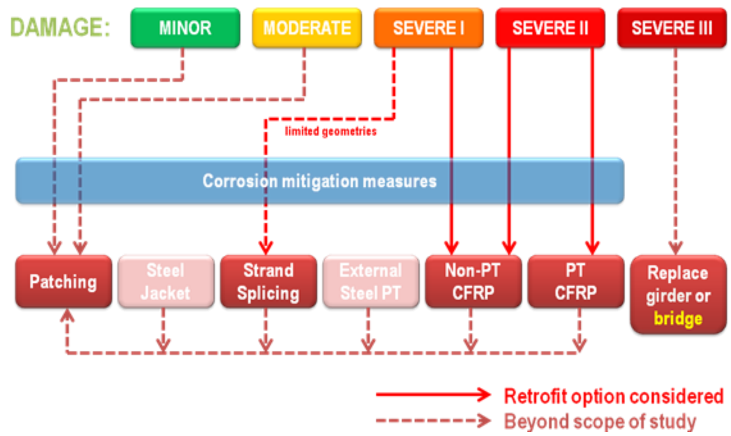
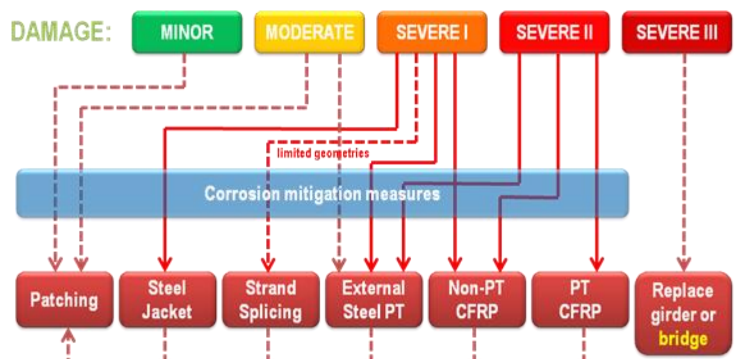


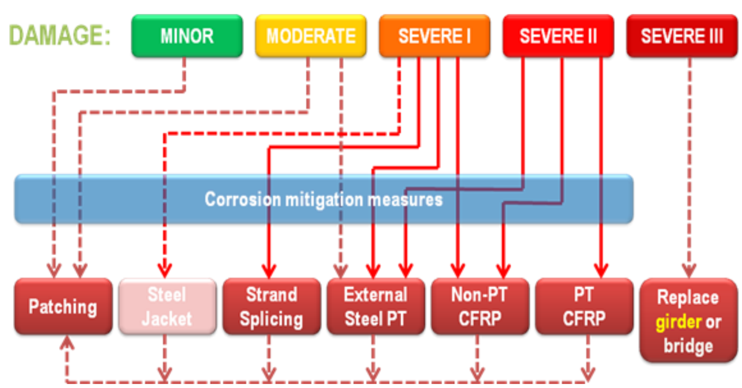
Figure 5.3 Prototype IB girder cross section.



(a) Adjacent box girders

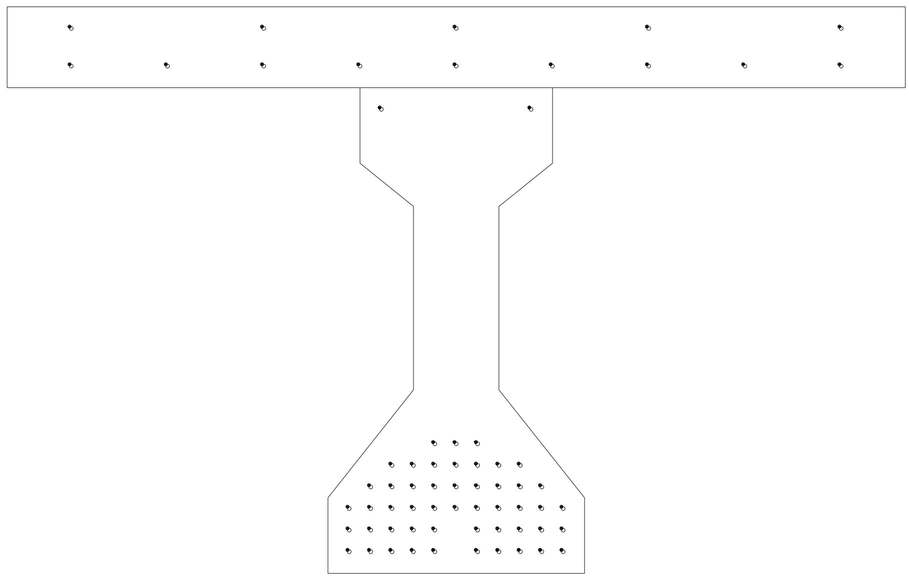


(b) Multi-box beam

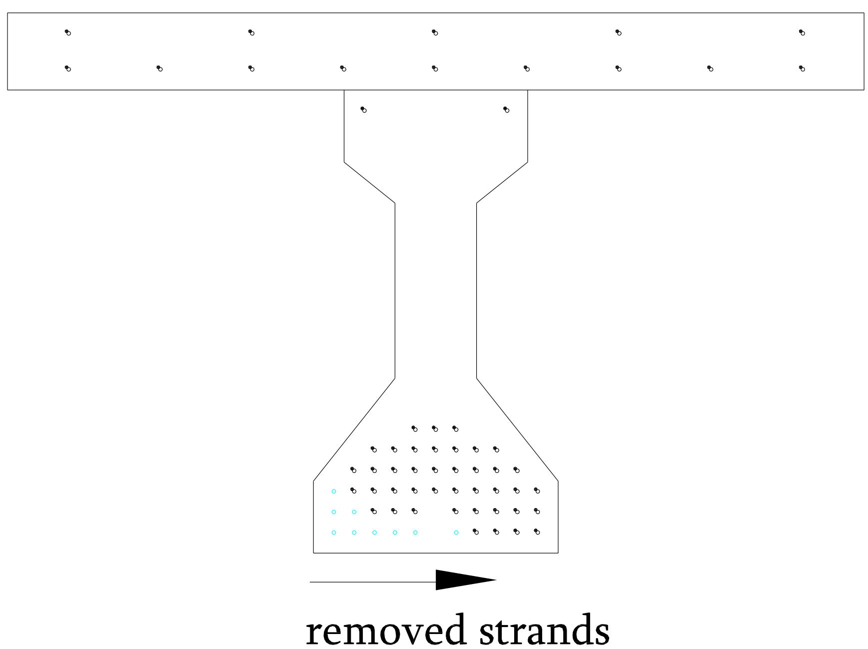


(c) I-beam

Figure 5.4 Flow charts illustrating viable retrofit techniques based on level of damage.

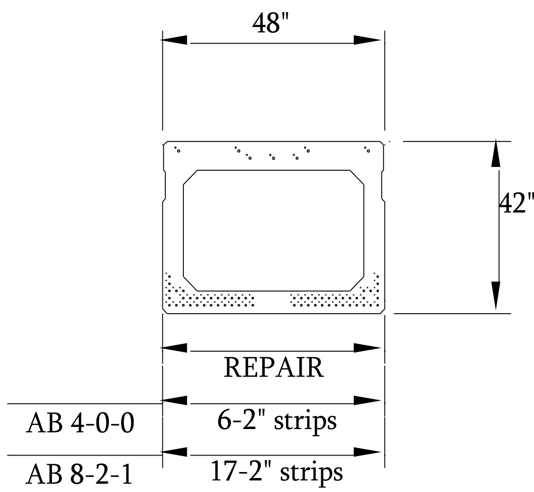


(a) case IB 0-0-0.

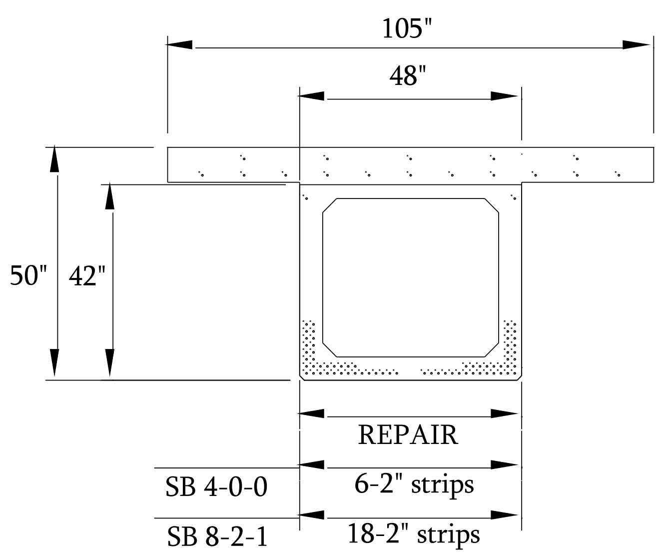


(b) case IB 6-2-1.

Figure 5.5 Example of analysis identification.



(a) AB CFRP strip repair.



(b) SB CFRP strip repair.

Figure 5.6 Non prestressed preform CFRP strip repairs.

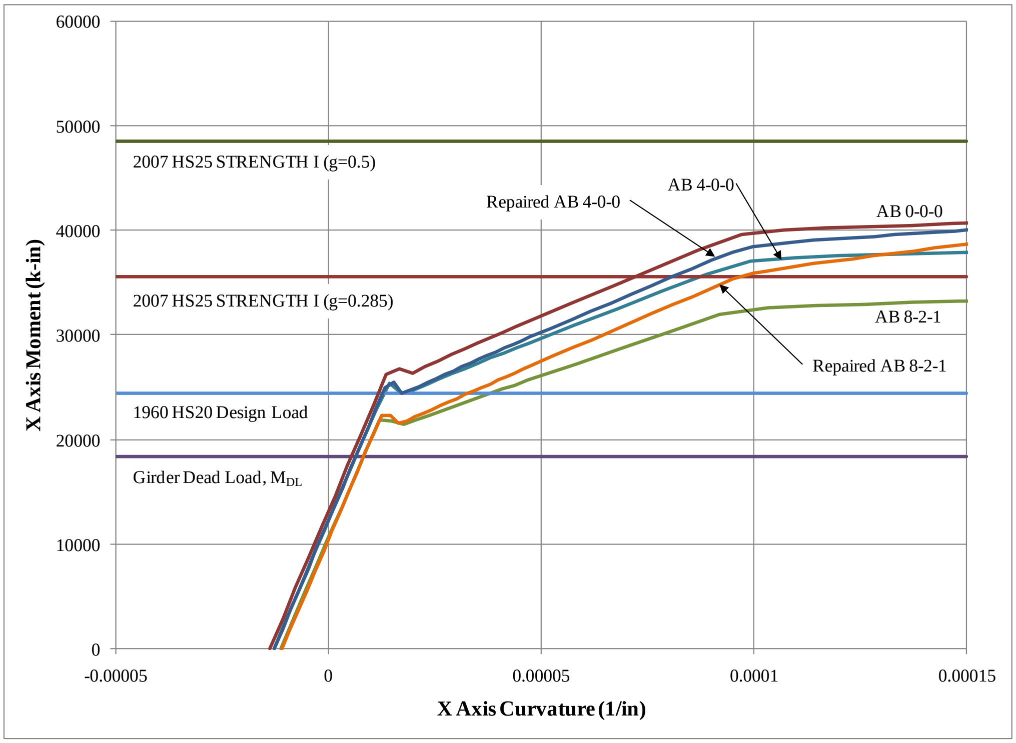


Figure 5.7 Preform CFRP strip repaired AB moment-curvature plot.

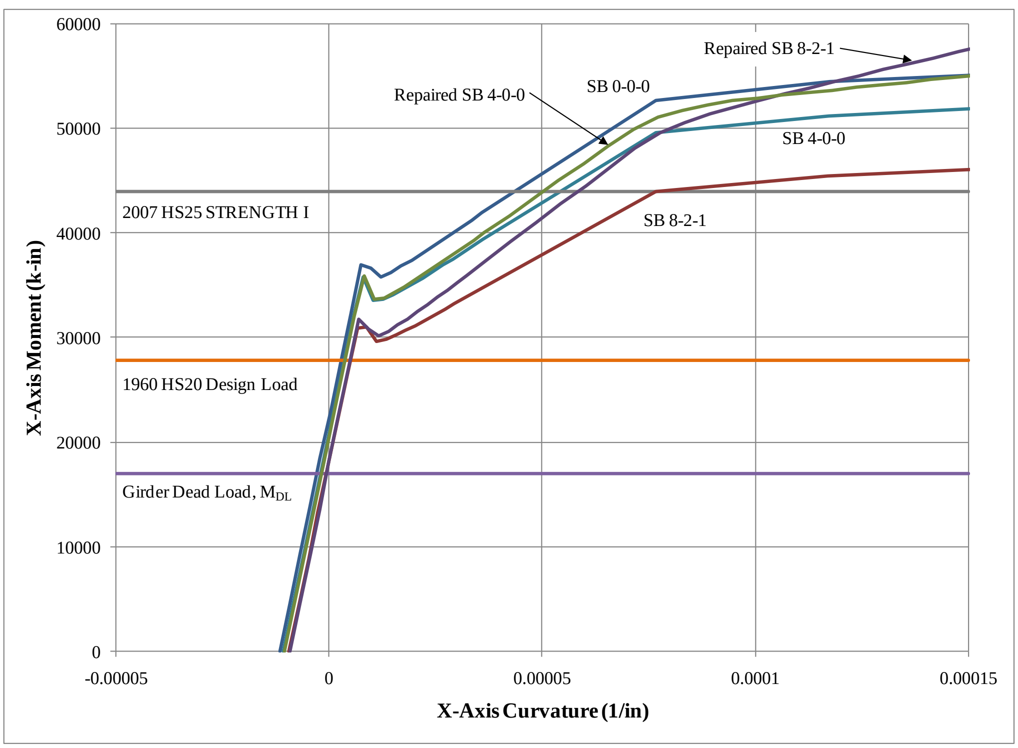


Figure 5.8 Preform CFRP strip repaired SB moment-curvature plot.

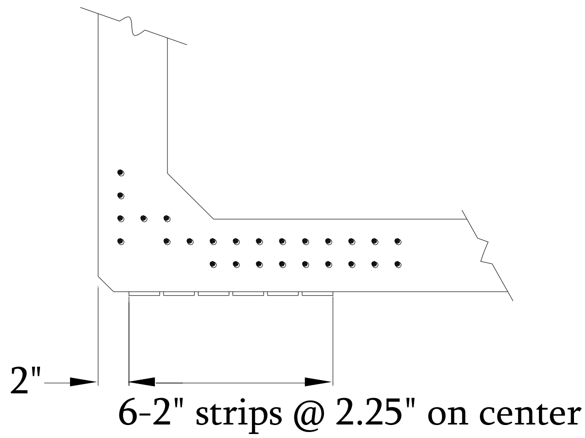


Figure 5.9 Suggested strip location for AB 4-0-0.

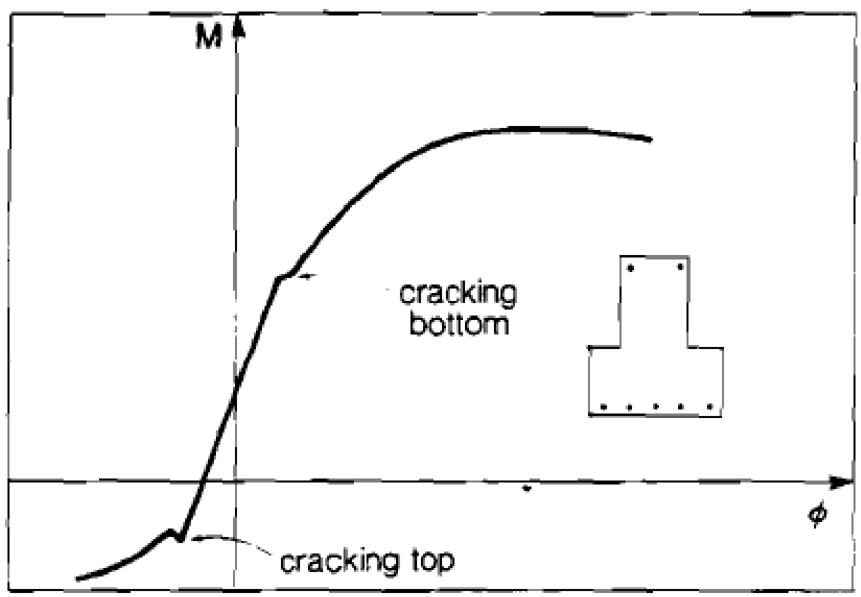


Figure 5.10 Flexural behavior of prestressed girders (Collins and Mitchell 1997).

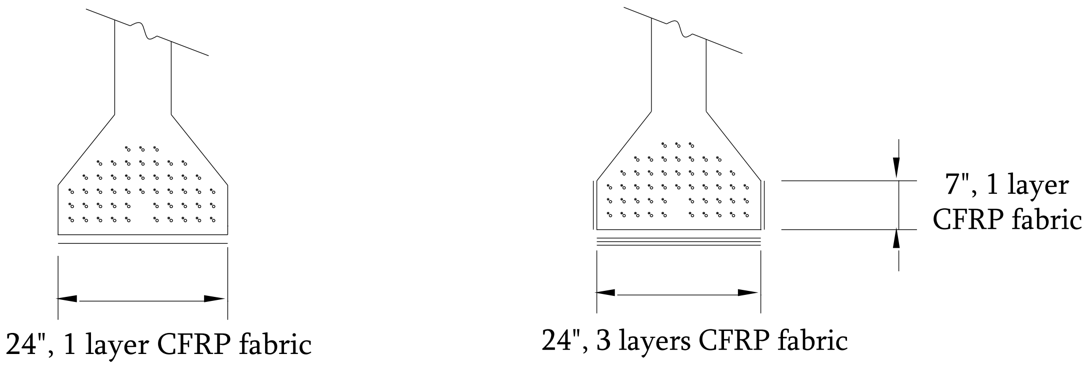


Figure 5.11 CFRP fabric repairs.

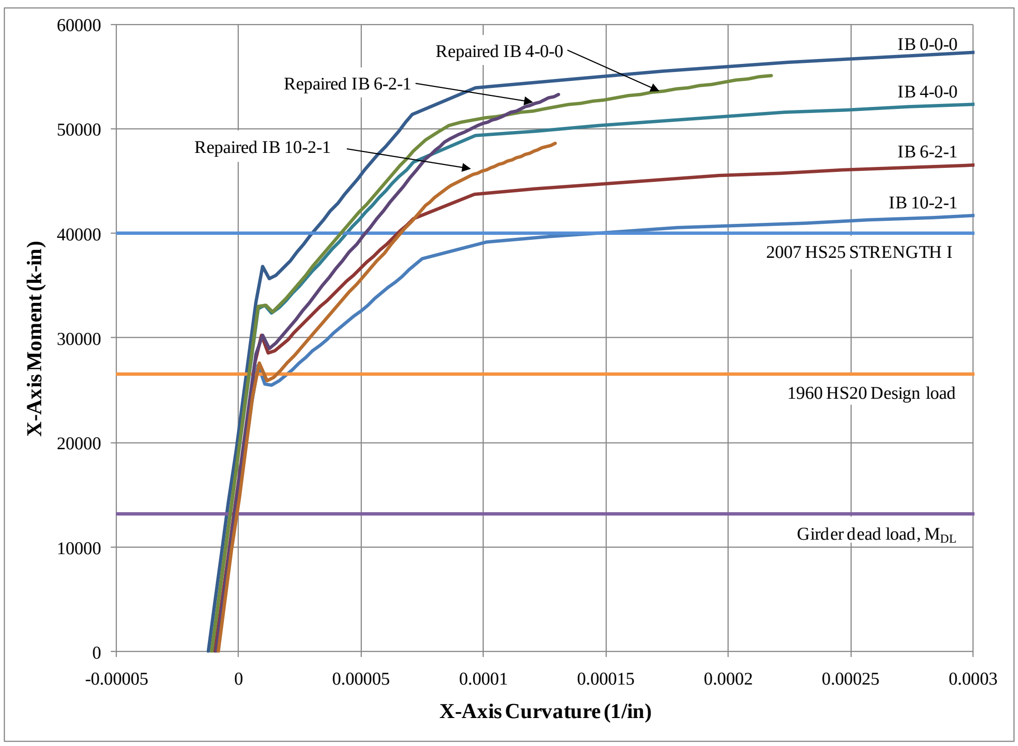


Figure 5.12 CFRP fabric repair moment-curvature plot.

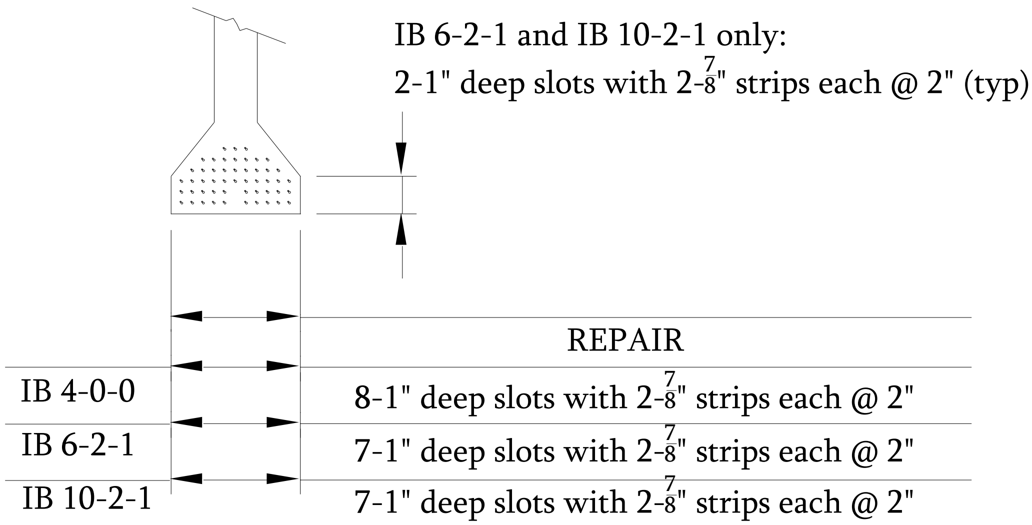


Figure 5.13 NSM repairs.

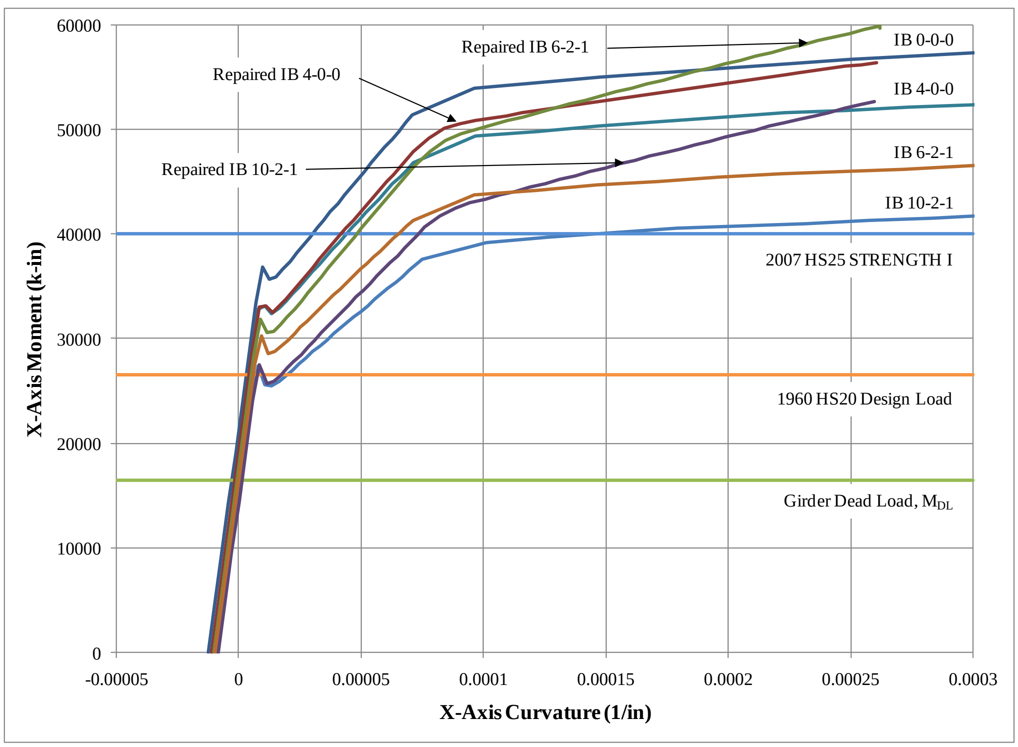


Figure 5.14 NSM repair moment-curvature plot.

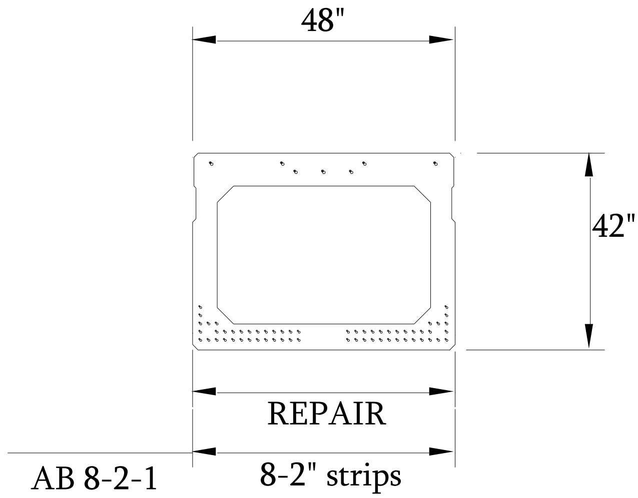


Figure 5.15 Prestressed CFRP repaired AB.

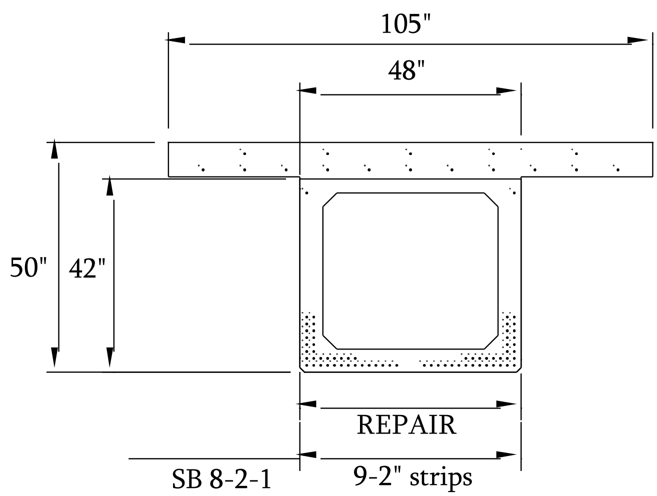


Figure 5.16 Prestressed CFRP repaired SB.

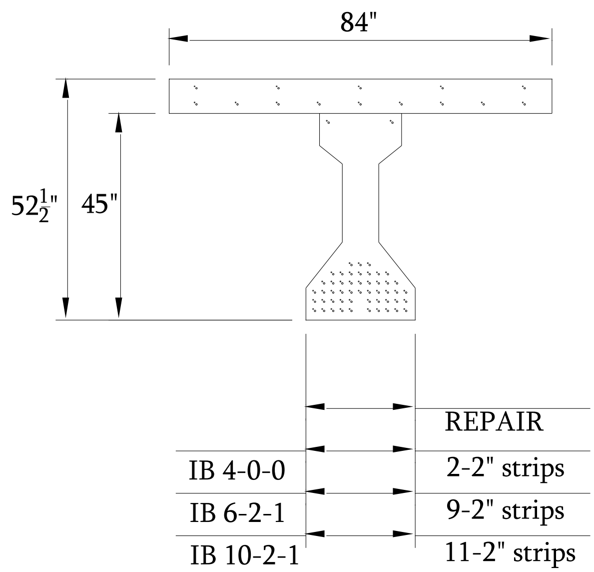


Figure 5.17 Prestressed CFRP repaired IB.

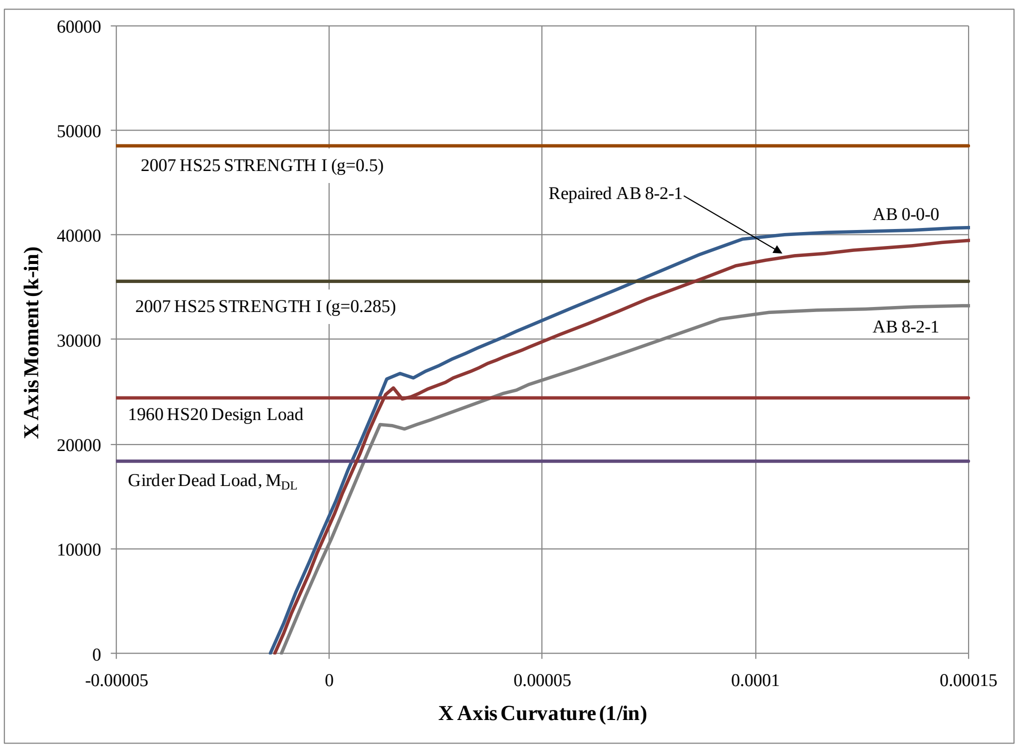


Figure 5.18 Prestressed CFRP repaired AB moment-curvature plot.

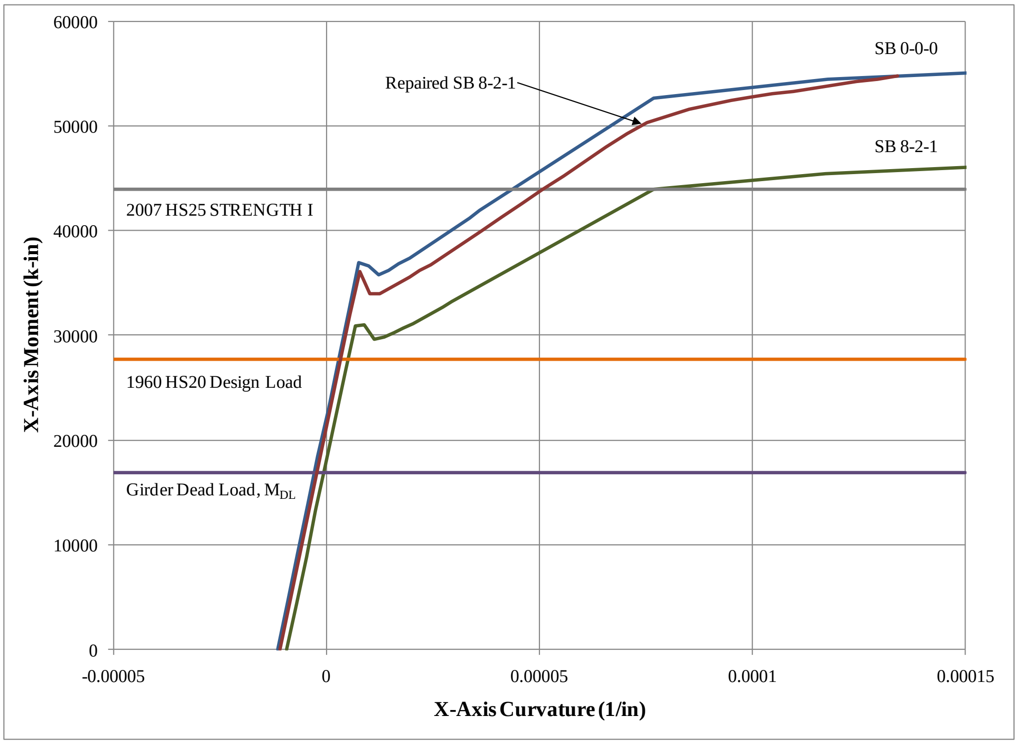


Figure 5.19 Prestressed CFRP repaired SB moment-curvature plot.

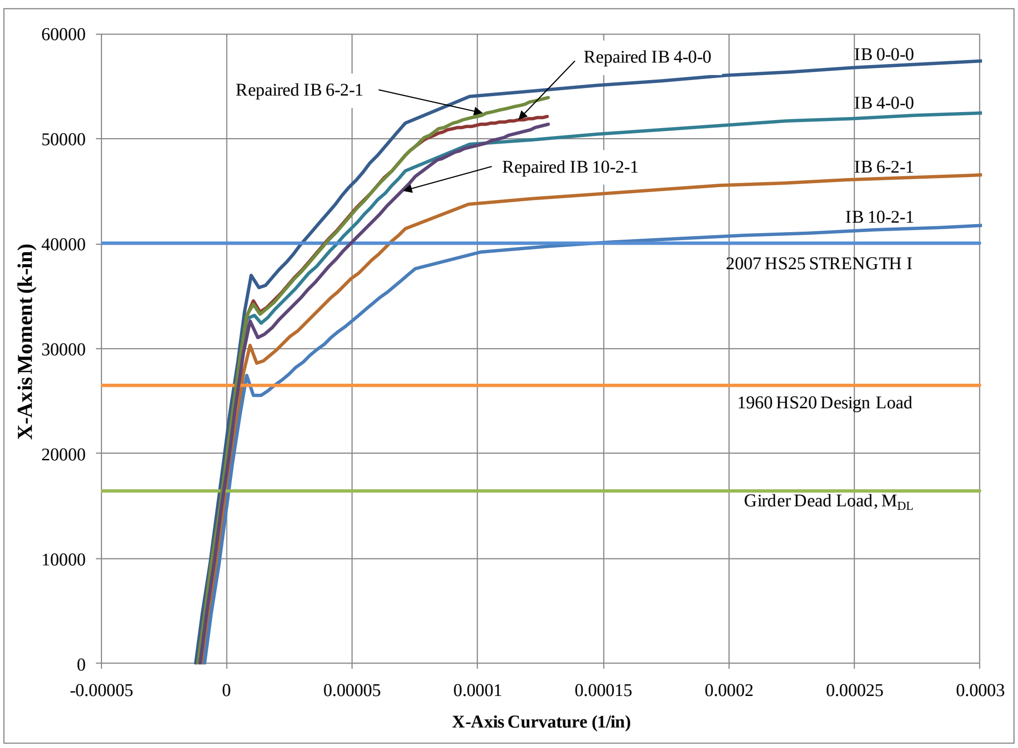


Figure 5.20 Prestressed CFRP repaired IB moment-curvature plot.

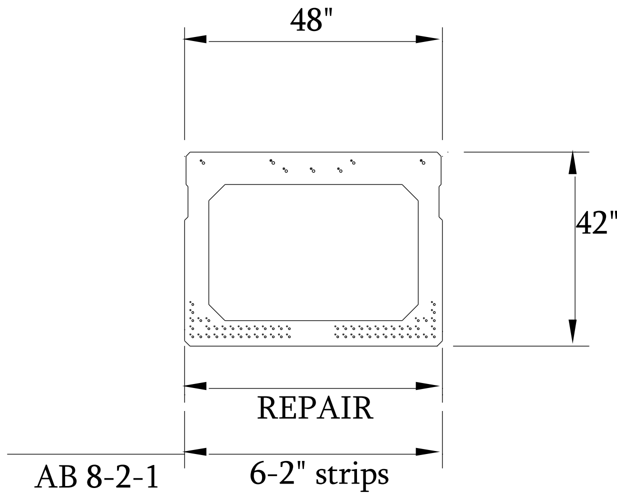


Figure 5.21 Post-tensioned CFRP repaired AB.

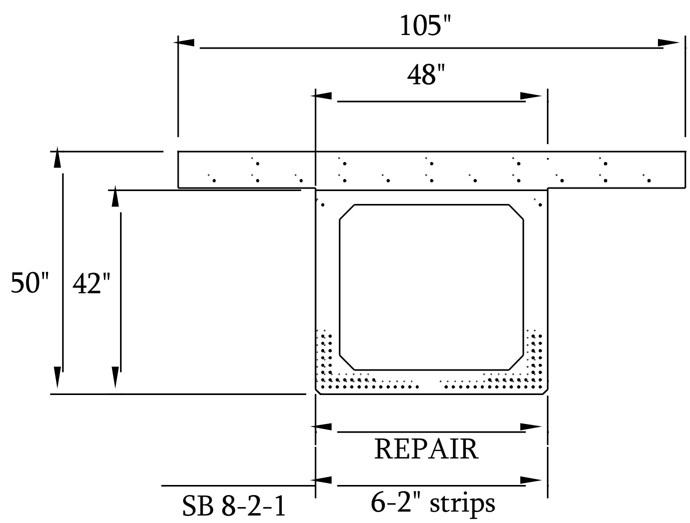


Figure 5.22 Post-tensioned CFRP repaired SB.

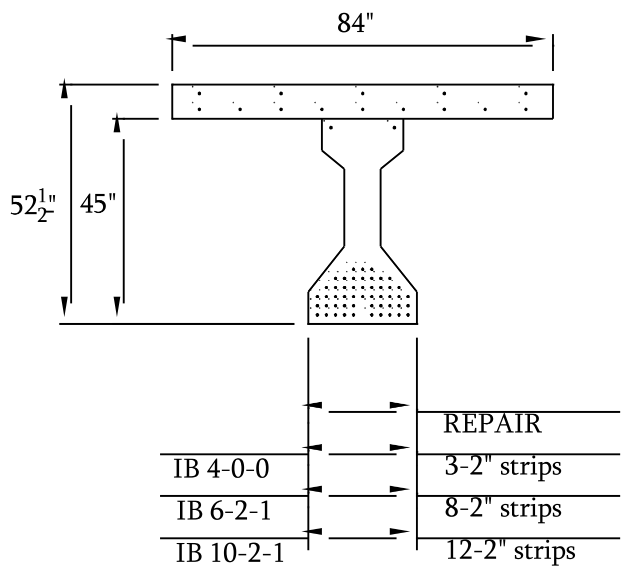


Figure 5.23 Post-tensioned CFRP repaired IB.

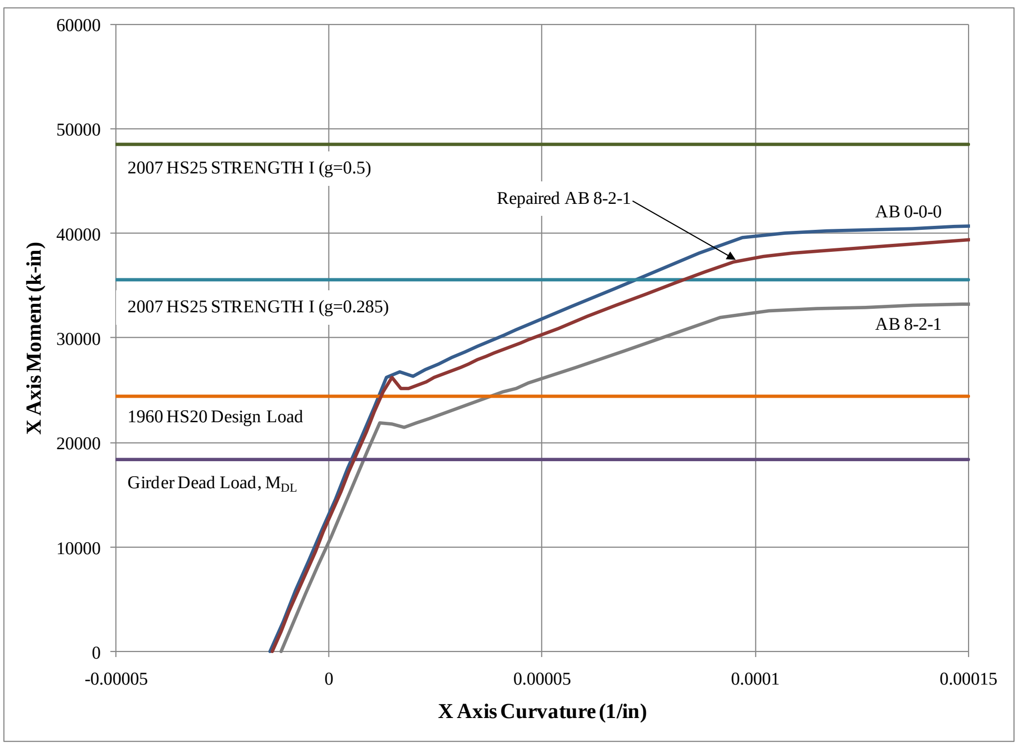


Figure 5.24 Post-tensioned CFRP repaired AB moment-curvature plot.

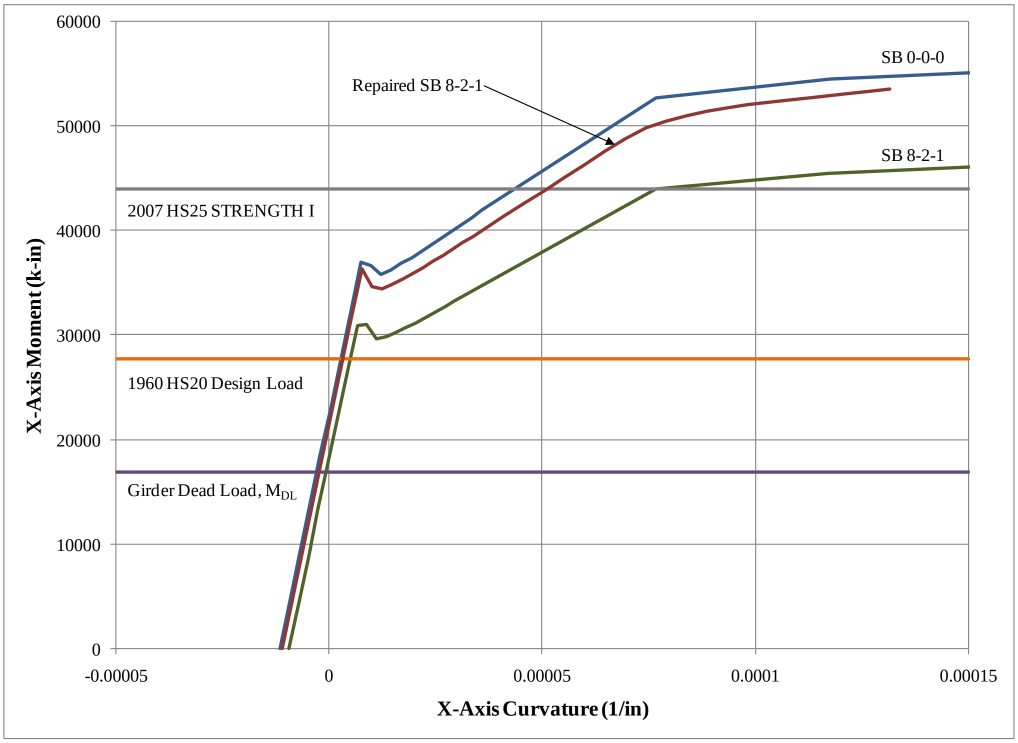


Figure 5.25 Post-tensioned CFRP repaired SB moment-curvature plot.

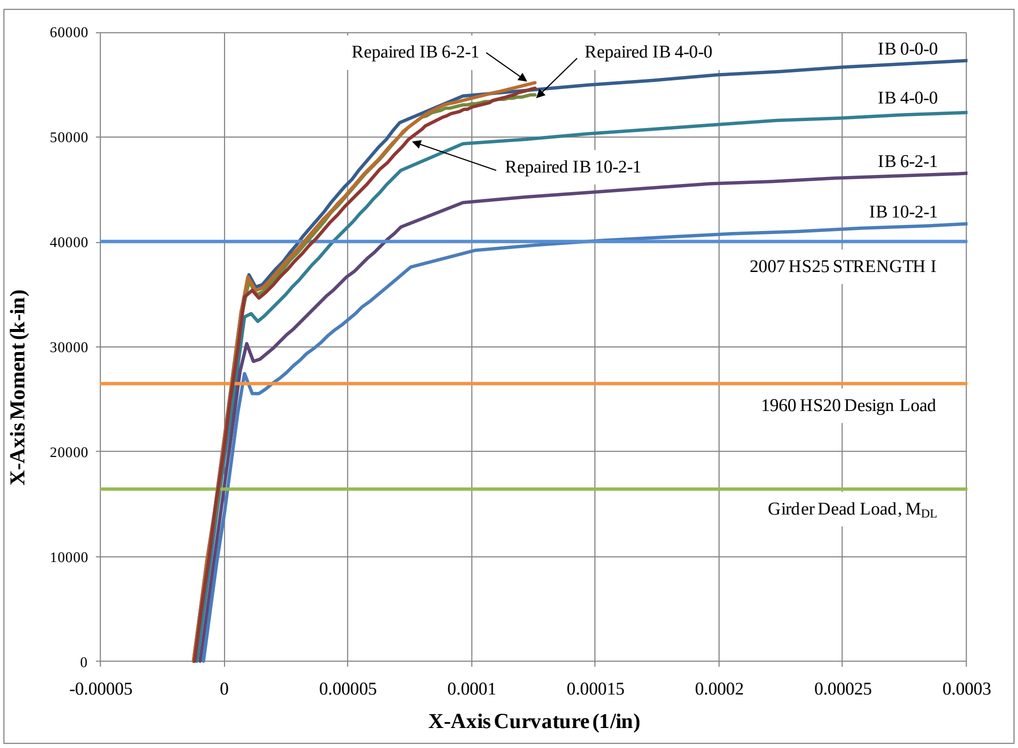


Figure 5.26 Post-tensioned CFRP repaired IB moment-curvature plot.

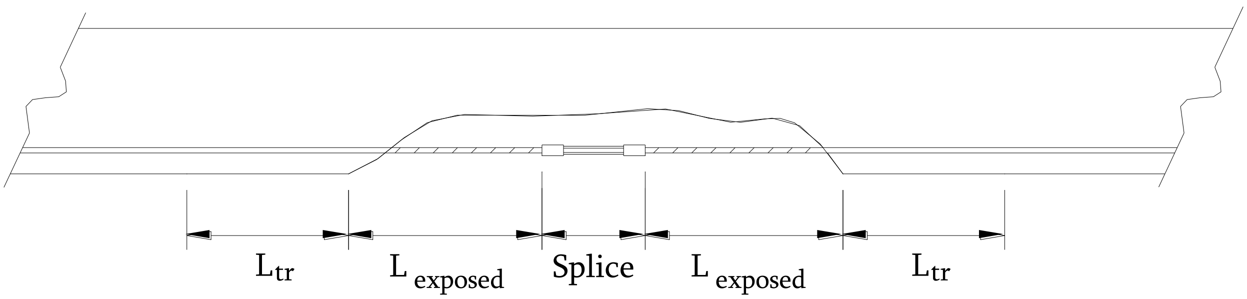


Figure 5.27 Schematic of quantities for Equation 5-1.

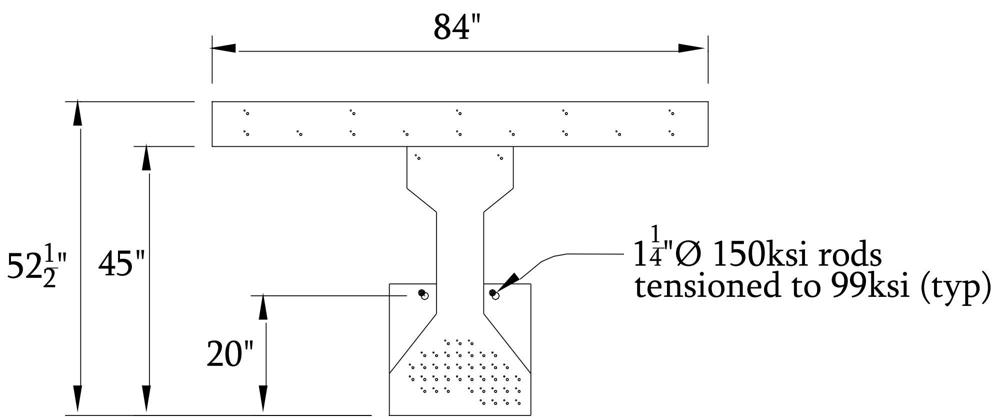


Figure 5.28 External post-tensioned steel repaired IB 6-2-1 drawing.

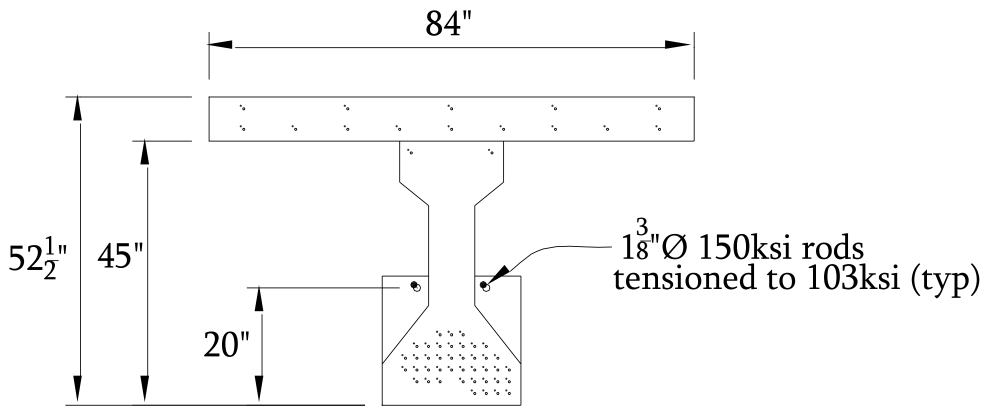


Figure 5.29 External post-tensioned steel repaired IB 10-2-1 drawing.

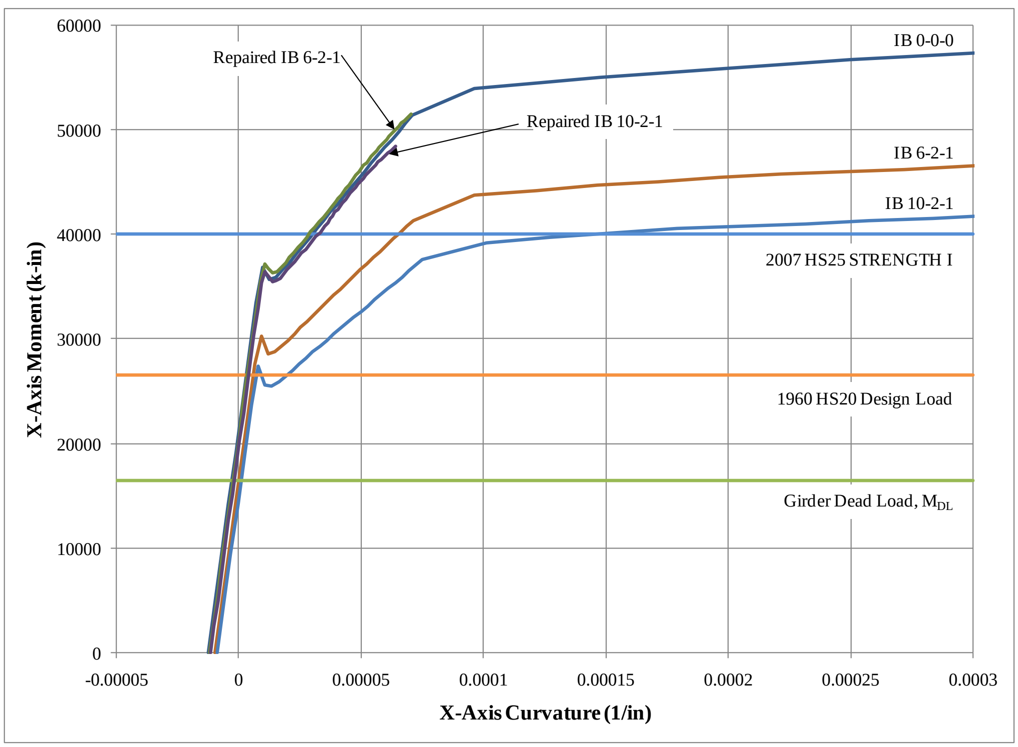
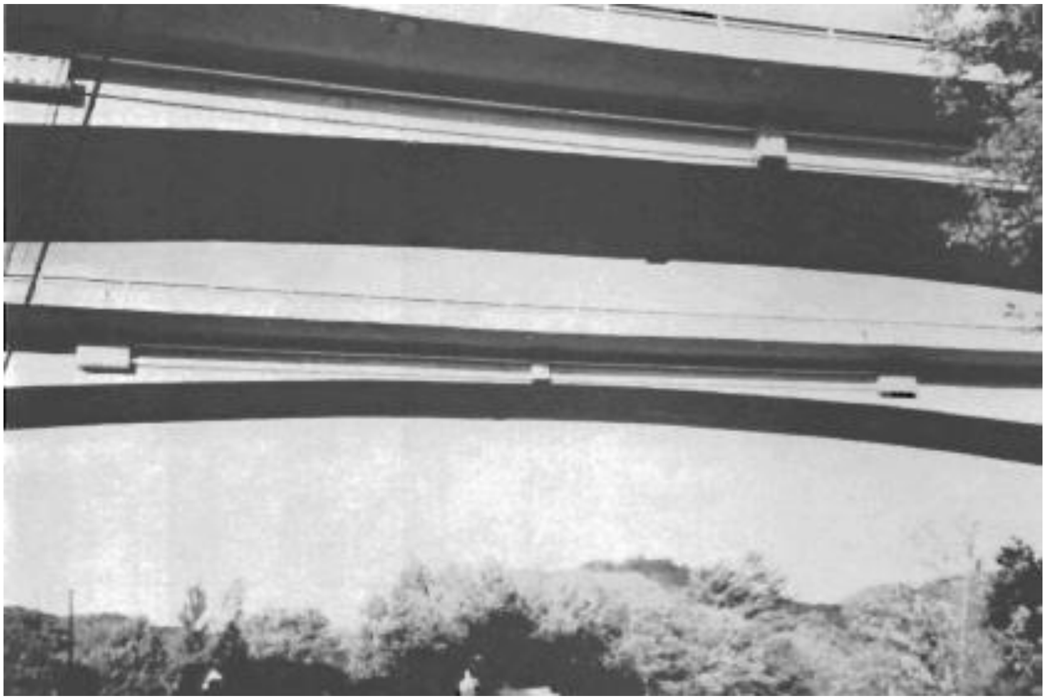
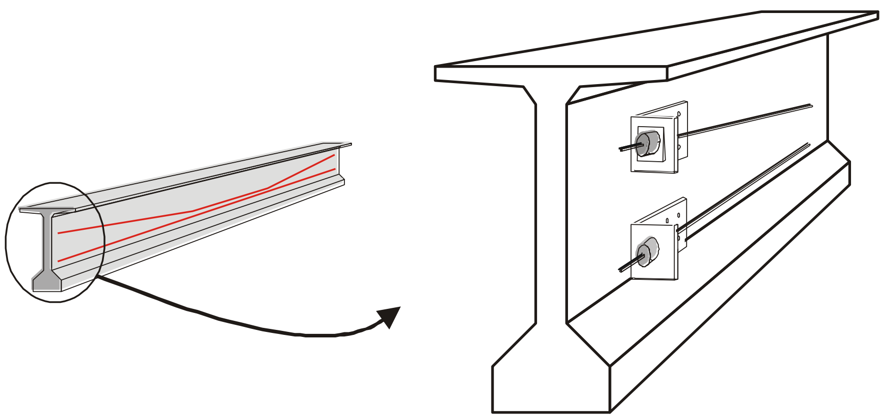


Figure 5.30 External post-tensioned steel repaired IB moment-curvature plot.



(a) Post-tension tendon retrofit with concrete bolsters (Collins and Mitchell 1997).



(b) steel angle anchorages for straight or harped strands.

Figure 5.31 Bolster examples.

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TASK 6: BEST PRACTICES RECOMMENDATIONS

This section presents recommendations made to the Pennsylvania Department of Transportation based on the findings of the reported investigation. Recommendations are summarized in Section 6.4. The basis of these recommendations are the findings of a parametric study evaluating 22 prototype repair designs for three prototype prestressed concrete highway bridge girder shapes: adjacent boxes (AB), spread boxes (SB), and AASHTO-type I-girders (IB), having four different damage levels. Although not applicable to all structure types or all damage levels, the repair techniques covered include the use of carbon fiber reinforced polymer (CFRP) strips, CFRP fabric, near-surface mounted (NSM) CFRP, prestressed CFRP, post-tensioned CFRP, strand splicing and external steel post-tensioning.

Since the girders have only been considered individually, no interaction effects between girders are considered and the global behavior of the bridge cannot be specifically addressed. It is important to reiterate that the nature of the repair technique is not generally affected by whether the structure is composite or non-composite as well as continuous or simple.

Recommendations are organized under three general topics:

- Bridge inspection
- Bridge rating and assessment
- Repair design selection criteria

6.1 Bridge Inspection Techniques

First and foremost, it is recommended that the inspection phase be separated from the assessment phase (Shanafelt and Horn 1980 and Harries 2006). The structure inspection should identify areas of damage or concern and report these quantitatively (if possible) or qualitatively (where necessary). Topics to be covered in the inspection report should include, but not be limited to the following (adapted and expanded upon from Shanafelt and Horn 1980):

1. Bridge name.
2. Bridge location description and location map.
3. Member type/identification.
4. Date of inspection.
5. Damage to prestressing elements.
    a. Exposed.
    b. Nicked.
    c. Gouged.
    d. Deformed.
    e. Severed.
    f. Damage locations.
    g. Narrative description of damage.
6. Damage to concrete.
    a. Spalls, nicks, scrapes and gouges (estimate sizes).
    b. Cracks including length, width and configuration (i.e. longitudinal, angled etc.).
    c. Reduction of concrete cover.
    d. Loose concrete.
    e. Damaged concrete.
    f. Shattered concrete.
    g. Stained or discolored concrete (i.e. from rust or efflorescence).
    h. Damage locations.

i. Narrative description of damage.
7. Extent of shear key (for AB girders only).
8. Poor/failing drainage systems.
9. Member displacement.
    a. Lateral or vertical.
    b. Rotation or twist.
    c. Camber (if measureable).
    d. Narrative description.
10. Date and cause of damage (if applicable).
11. Law enforcement accident report (if applicable).
12. Damaged member category.
    a. Fracture critical.
    b. Primary, secondary etc.
13. Site conditions.
    a. Damage over water, traffic etc.
    b. Factors affecting repair solutions.
14. Roadway clearance.
15. Photographs.
16. Differences between previous inspection report and current report (identify new conditions).

Equally significant, the manner in which each structure is inspected should be standard so that each structure is judged with the same criteria. As a result, a standardized inspection procedure should be developed and should include proper inspection techniques, tools and forms. The standardized inspection should allow for quality, relevant information regarding the structure's quality to be translated from the inspection to the engineer.

PennDOT document P-800 *Instructions for Acceptance of Prestressed Beams with Cracks. Crack Classification* (PADoH 1970) provides a degree of guidance in identifying cracks in prestressed members. However it must be noted that this document is intended for establishing acceptance of new, as-delivered prestressed girders and therefore focuses on fabrication, handling and erection-related damage only. P-800 does not address the effects of significant impact, degradation or aging. The PCI *Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products* (PCI MNL-137-06) provides more detailed guidance in evaluating the nature, degree and source of damage to prestressed girders including damage related to use and aging of the structure.

6.2 Bridge Rating and Assessment

Load rating and structure capacity assessments should be conducted by an engineer as a task separate from the inspection (Shanafelt and Horn 1980 and Harries 2006). The information provided by the inspection should be of such quality to allow the engineer to properly assess the structure's strength. Section 1 of this report presents a detailed description of observed damage to prestressed concrete bridge members in Southwestern Pennsylvania. NCHRP *Report 226* (Shanafelt and Horn 1980) provides guidance for the assessment of damaged prestressed concrete bridge girders. Additionally, Harries (2006) provides a guide for inspecting such girders and identifying and assessing damage types. An example of damage assessment guidance that should be emphasized is PennDOT's adoption of the '150% rule' for assessing the area of lost prestressing strand: [paraphrasing] *when assessing corrosion damage to a prestressed concrete girder, the area of prestressing strand assumed to be ineffective due to corrosion shall be taken as 150% of that determined by visual inspection*. This guidance, recommended by Harries (2006) and Naito et al. (2006) is believed to conservatively capture the unseen (uninspectable) corrosion of strands adjacent to those damaged by corrosion.

In general, the use of plane sections analysis using standard Whitney stress block factors has been shown to be adequate for assessing the capacity of damaged and repaired girders. Harries (2006) describes some limitations of a plane sections approach for beams having highly eccentric loading or resistance. A parallel study (Russell 2009) has as its objective the simplification of highly eccentric sections such that a plane sections analysis approach may be utilized. In the present work, only sections having nominal eccentricities were considered. Harries (2006) has shown that these eccentricities have essentially no effect on the capacities derived using conventional plane sections analyses. Analysis of interior girders may safely neglect effects of eccentricities since the girders are ‘constrained against twist’ by adjacent girders.

It is assumed that broken strands do not contribute to the section capacity. Harries (2006) proposes that allowing such redevelopment of prestressed reinforcement should be considered on a case-by-case basis; little is understood of this behavior. For example, a ruptured strand near midspan may not effectively redevelop or if it does so it may not be effectively redeveloped for some distance from the rupture, at which point the applied loads are reduced. Conversely, a damaged strand near an abutment may be effectively redeveloped and able to contribute fully to the section resistance at midspan. A brief experimental study intended to address this effect is proposed in Section 7.2 of this report.

Based on the observed condition of the Lakeview Drive shear keys and the general inability to assess their soundness in an inspection, it is recommended that the presence of the shear keys be neglected in rating adjacent prestressed box girder bridges (Harries 2006). Thus:

1. The flexural axle distribution factor is taken as g = 0.5 for girders less than 72” wide.
2. The exterior girder dead load (DC and DW) and barrier wall loading (DC) is carried entirely by the exterior girder on which it rests.

The latter issue is believed to have contributed to the Lake View Drive Bridge collapse (Harries 2006).

A further issue, not addressed in the present study, is the likelihood of the exterior girder of an adjacent prestressed box girder bridge carrying live load. This will be affected by the girder width and the curb slab geometry. Regardless of geometry, a value of g = 0.5 remains appropriate regardless of loading condition:

1. If the exterior girder geometry cannot carry vehicular live load (LL), g = 0.5 is conservative. It would be reasonable to revert to the LRFD-prescribed (AASHTO 2007) calculation which will yield a value around g = 0.30. This latter calculation assumes that the shear transfer affected by the shear key is intact.
2. If the geometry dictates that the exterior girder is not *expected* to be loaded by vehicular live load (LL), but inadvertently does, the use of g = 0.5 is appropriate.
3. If the exterior girder is expected to carry vehicular live load (assumed to be a single side of a vehicle), again g = 0.5 is appropriate. If it can be shown that the shear keys are sound and transmitting forces between girders are expected, distribution factors may be calculated in the traditional manner (AASHTO 2007).

6.3 Repair Design Selection Criteria

The matrices shown in Figure 5.4 present a range of viable repairs for each girder type and do not consider the specific damage level. Nonetheless, the damage level dictates which repair method can be used. For example, in an IB girder, strand splicing is a potential repair approach, but only if a few strands need to be replaced. The geometry of the strand arrangement and strand splice make this method impractical for heavier damage. Although 'percentage of strands lost' appears to be a representative indicator of girder strength, the only correlation found between percentage of lost strands to repair method has been at the level of 25% of strands lost. At this level of damage, repair (restoration of undamaged capacity) becomes impractical (as seen in the case of IB 10-2-1). This is not to say that the girder cannot be repaired, but the resources necessary to repair this girder would be significant and thus replacement may become a more attractive solution.

Often, the girders have been designed to have a specific stress level at the soffit. To restore this, an active repair (i.e. strand splicing, prestressed or post-tensioned repairs) should be selected so that as much of the prestressing force is restored as possible. However, when soffit stress is not the main consideration, any of the described techniques, active or passive, may be used. Current best-practices provide that non-prestressed repair methods are preferred for the ease of installation. As is shown throughout Section 5, the benefit obtained from prestressing CFRP is generally marginal. This opinion is shared by the authors of this report and a number of practitioners presently engaged in the type of retrofits described here.

The repair type chosen must be done so on a project-by-project basis. At this point, it is not feasible to standardize repair type selection based on damage level due to the variability between structures, the unique nature of damage to a particular girder and the original girder's design or stress requirements. Nonetheless, Figure 5.4 provides a summary of viable repair techniques for each scenario and some additional guidelines (rules of thumb) are presented in the following sections.

The need to mitigate active corrosion as part of any repair scheme is emphasized. First and foremost, the source of corrosion must be mitigated (leaking deck drains, salt spray, etc.). Once the source is removed, removal of all corroded strand and chloride contaminated concrete followed by patching represents the typical protocol. For large scale chloride contamination, chloride extraction techniques are available. Passive corrosion mitigation techniques such as the inclusion of sacrificial anodes should be considered as a complement to sound patching techniques. Active corrosion mitigation measures such as impressed current cathodic protection are presently **not** recommended for prestressed concrete structures due to the sensitivity of high strength strand to embrittlement.

6.3.1 Repair Technique Applicability

The repair method chosen is a function of the original girder's design considerations such as soffit stress (Preston et al. 1987), girder shape, strand spacing or layout and damage, amongst other factors. Also, the goal of the repair must be considered, i.e. if the repair must restore prestressing force (an active repair) or flexural capacity (achievable with a passive repair). Table 6.1 summarizes the potential applications and a number of selection and design considerations for each repair type. Although specific damage levels are not suggested, this table suggests the limits of applicability of each repair type. Table 6.1 updates and revises the performance comparison matrix presented by Shanafelt and Horn (1980) and presented in Table 3.1. Due to the different bases for comparison (inclusion of CFRP methods), the ranking and practicality of various methods reported by Shanafelt and Horn have changed. For instance, steel jackets are not considered practical. They are cumbersome, untested, and their design, installation and performance are all expected to be exceeded by CFRP methods. While strand splicing is felt to viable for localized repairs associated with individual impacts, this method is limited by the degree of damage it can reasonably mitigate.

In terms of CFRP methods, non-prestressed methods are well established in both the literature and practice (Section 3). Prestressed or post-tensioned methods are presently limited to proprietary systems and have similarly limited field experience. Nonetheless, post-tensioned CFRP holds great promise for highway bridge applications. NSM CFRP out performs surface-mounted CFRP, however this performance comes at a cost in terms of constructability. Additionally, NSM repairs may be more limited than surface mounted methods due to slot geometry and spacing requirements.

All external methods require protection from the environment. Steel methods may use galvanizing, epoxy coating or encased (unbonded post-tensioning type) strand. CFRP itself requires little environmental protection, although adhesive systems do. Therefore, CFRP systems are often painted with a gel coat to limit moisture intrusion and protect against UV radiation.

External repair methods must also be protected from mechanical damage. Repairs that are attached to the beam soffit encroach upon the roadway clearance below. The only viable method for protecting against mechanical damage is ensuring the repair is not impacted. This therefore, should be an initial design consideration. In general, external CFRP systems are smaller and have a 'lower profile' than steel systems. NSM and strand splicing are internal repairs and have little effect on beam geometry.

Cost and aesthetic rankings given in Table 6.1 are quantitative assessments of the authors. Once again, due to the unique nature of each repair project, it is difficult to provide cost efficiency in a general sense.

6.3.2 Girder Shape

Girder shape plays a role in repair selection and design. For instance, IB girders have a more vertically distributed arrangement of strands resulting in a higher center of gravity of strands than AB and SB girders. As a result, strands lost on the bottom row in an IB girder have a greater proportional affect on the strand center of gravity (and thus girder capacity) when compared to the same damage for an AB or SB girder. That is, one lost strand has more of an impact on the flexural capacity in an IB girder than for an AB or SB girder. As a result, the repairs for IB girders are more substantial as compared to those for AB or SB girders having the same damage level. Furthermore, the bulb of an IB girder results in certain geometric constraints on the repair: NSM slots are limited and external CFRP requires rounding of the bottom corners in order to be extended up the side of the bulb. Extending the CFRP vertically from the soffit also results in proportionally less efficient use of the CFRP (as its centroid rises).

6.3.3 Repair Ductility

Using ultimate curvatures as an indicator of ductility, it can be seen that passive repair methods are more ductile than active methods. It is believed that the active utilization of the material (i.e. post-tensioning) creates a greater possibility of material yielding and thus a less ductile failure than a passive repair application. As a result, it is concluded that maximizing an active repair for a girder is not ideal and other solutions should be investigated. One possibility not considered here is a 'partially prestressed' repair where only a portion of the CFRP provided is post-tensioned.

6.3.4 Design of Repair and Assessment of Repair Capacity

Section 5 of this report demonstrates through 22 prototype repair examples, the design and capacity assessment of various prestressed repair methods for a range of degrees of damage. While, no broad classifications have been presented directly linking damage level (or a range of damage) to specific repair types, it is concluded that when 25% of the strands in a girder no longer contribute to its capacity, girder replacement is a more appropriate solution. Beyond this, consideration must be given to the objectives of an individual repair. In the presented examples, the objectives of all repairs was to restore the undamaged capacity of the damaged girder. In some instances this was not practical. Despite some repairs failing to achieve their target capacities, the behavior of all examples was improved. This leads to three possible scenarios with respect to assessment of the repair capacity:

1. The target capacity is achieved and the repair is considered successful.
2. The target capacity is not achieved; however the beam behavior is improved sufficiently to carry required loads. The corollary of this case is that the target capacity is selected only at a level to allow the beam to perform adequately, but not necessarily achieve its original undamaged capacity. That is: the target capacity was selected only as high as is necessary to provide adequate performance.
3. The target capacity is not achieved and the beam behavior is not improved sufficiently. In this case an alternate repair method or beam replacement is required. This case permits the limit of each repair method to be assessed.

NCHRP Report 514, *Bonded Repair and Retrofit of Concrete Structures Using FRP Composites: Recommended Construction Specifications and Process Control Manual* (Mirmiran et al. 2004), and NCHRP Report 609, *Recommended Construction Specifications and Process Control Manual for Repair and Retrofit of Concrete Structures Using Bonded FRP Composites* (Mirmiran et al. 2008), provide guidance for application, process control, handling and specifying bonded FRP repair and retrofit systems for concrete structures .

PCI *Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products* (PCI MNL-137-06) provides guidance for concrete patching appropriate to repair mild or moderate levels of damage.

6.4 Summary of Recommendations

The following summarizes the recommendations made based on the findings of the reported investigation.

6.4.1 Inspection of Prestressed Concrete Bridge Girders

1. Inspection is conducted as a task separate from assessment.
2. Identify all damage to prestressing elements.
3. Identify all damage to concrete.
4. Determine extent of shear key (when applicable).
5. Identify member deflections and camber, if possible.
6. Identify site conditions that may affect repair solution and technique.
7. Determine/verify roadway clearance (when applicable).

6.4.2 Assessment of Prestressed Concrete Bridge Girders

1. Load rating and structure assessment should be conducted by the responsible engineer.
2. The use of the ‘150% rule’ should be adopted and employed.
3. Plane sections analysis using standard Whitney stress block factors is adequate.
4. Capacity of eccentrically loaded girders should be considered for completeness (the work is not covered here, Russell 2009 discusses this topic).

5. It is conservative to neglect the ‘redevelopment’ of a severed strand some distance from the damage and thus remove the strand from any analysis. Case-by-case assessment using appropriate engineering judgment is necessary in this regard, however.
6. For AB girders only (Harries 2006):
    a. The flexural distribution factor is taken as g = 0.5 for girders less than 72” wide.
    b. The exterior girder dead load and barrier wall loading is carried entirely by the exterior girder on which it rests (i.e.: no distribution).
    c. Consider the effects that the previous recommendations have on the live load distributed to the exterior girder. If no live load is anticipated, it may be appropriate to assume some distribution from the first interior girder. AASHTO LRFD (2007) calculations are appropriate in this case.

6.4.3 Repair Design Selection Criteria

1. No correlation between damage and repair type is proposed except that girder replacement is likely required at a loss of 25% of strands.
2. Original design criteria and soffit stress requirements may dictate an active or passive repair.
3. Repair type should be selected on a case-by-case basis.
4. Strands lost in girders with a more vertically distributed strand arrangement have a greater impact on flexural capacity as compared to girders with a more horizontal strand layout (i.e. IB girders as compared to AB or SB girders).
5. Passive repair techniques display a more ductile behavior than active repair techniques.
6. Construction specifications for CFRP systems may be based upon the recommendations of NCHRP Report 609 (Mirmiran et al. 2008).

Speed of mobilization (i.e. material preparation or item fabrication) along with constructability and speed of repair all must be considered in repair type selection. These criteria are directly related to the viability and cost of a repair technique.

The specifications contained in NCHRP Report 514, *Bonded Repair and Retrofit of Concrete Structures Using FRP Composites: Recommended Construction Specifications and Process Control Manual* (Mirmiran et al. 2004), and NCHRP Report 609, *Recommended Construction Specifications and Process Control Manual for Repair and Retrofit of Concrete Structures Using Bonded FRP Composites* (Mirmiran et al. 2008) should be adopted for repairs involving these materials. These specifications represent the best available practices and have been approved by FHWA in this regard.

Table 6.1 Repair Selection Criteria.

Damage Assessment Factor	Repair Method								
	preform CFRP strips	CFRP fabric	NSM CFRP	prestressed CFRP	PT CFRP	PT steel	Strand Splicing	Steel Jacket[1]	Replace Girder
Damage that may be repaired	Severe I	low Severe I	Severe I	Severe II	Severe II	Severe II	low Severe I	Severe II	Severe III
Active or Passive repair	passive	passive	passive	marginally active	active	active	active or passive	passive or marginally active	n/a
Applicable beam shapes	all	all	IB, limited otherwise	all	all	all	IB, limited otherwise	IB	all
Behavior at ultimate load	excellent	excellent	excellent	excellent	excellent	excellent	excellent	uncertain	excellent
Resistance to overload	limited by bond	limited by bond	good	limited by bond	good	excellent	excellent	uncertain	excellent
Fatigue	limited by bond[2]	limited by bond[2]	good	limited by bond[2]	excellent (unbonded)	excellent	poor	uncertain	excellent
Adding strength to non-damaged girders	excellent	good	excellent	excellent	excellent	excellent	n/a	excellent	n/a
Combining splice methods	possible	possible	unlikely	possible	good (unbonded)	good	excellent	excellent	n/a
Number of strands spliced	up to 25%	limited	limited by slot geometry	up to 25%	up to 25%	up to 25%	few strands	up to 25%	unlimited
Preload for repair[3]	no	no	no	no	no	no	possibly	possibly	n/a
Preload for patch[3]	possibly	no	yes	possibly	possibly	possibly	yes	no	n/a
Restore loss of concrete	patch prior to repair	patch prior to repair	patch prior to repair	patch prior to repair	patch prior to repair	patch prior to repair	excellent	patch prior to repair	n/a
Speed of Mobilization	fast	fast	moderate	moderate	moderate	moderate	fast	slow	very slow
Constructability	easy	easy	difficult	difficult	moderate	moderate	difficult	very difficult	difficult
Speed of repair	fast	fast	moderate	moderate	moderate	moderate	fast	slow	very slow
Environmental impact of repair process	VOCs from adhesive	VOCs from adhesive	VOCs from adhesive & concrete sawing dust	VOCs from adhesive	minimal	minimal	minimal	welding	typical erection issues
Durability	requires environmental protection	requires environmental protection	excellent	requires environmental protection	requires environmental protection	requires corrosion protection	excellent	requires corrosion protection	excellent
Cost	low	low	moderate	moderate	moderate	low	very low	moderate	high
Aesthetics	excellent	excellent	excellent	excellent	fair	fair	excellent	excellent	excellent

n/a: not applicable

[1] 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.

[2] see Harries et al. (2006) for a discussion of fatigue of bonded CFRP repair systems.

[3] Preload may be required for the repair or simply to pre-compress associated concrete patches. Jackets render the need to pre-compress the patch unnecessary.

TASK 7: CANDIDATE DEMONSTRATION PROJECTS

The following descriptions of appropriate implementation/demonstration projects have been developed based on discussions between Kent Harries and Jarret Kasan (PITT), Lou Ruzzi (PennDOT) and Ahmad Ahmadi (SAI) and a site visit held November 7, 2008.

One exterior girder from each of two decommissioned bridges will be recovered and delivered to the Laboratory Hill Road site (I70 at Rte 40) used to test the Lake View Drive girders. The girders are as follows:

- S-3733A – 83' long prestressed 42 x 48 adjacent box girder
- S-3736A – 67' long prestressed 42 x 48 spread box girder

7.1 Proposed Research Program

Two projects are anticipated. **Project A** is intended only to provide test support for SAI/OSMOS. **Project B** will focus on the implementation/demonstration of appropriate repair methods for prestressed girders. This project should be engaged under the PITT/PennDOT IGA and will continue to involve SAI and OSMOS. Each is described briefly below.

Project A: PITT will provide test support for SAI project to provide evaluation of OSMOS fiber optic monitoring system. PITT will conduct the following tasks:

1. Design changes to existing test set up at Laboratory Hill Road site to accommodate shorter beams (Lake View Drive girders were tested over 85' span).
2. Oversee changes required from Task 1; SAI will contract these.
3. Oversee delivery and installation of test girders. SAI will contract this work.
4. Mobilize for pseudo-static testing of girders (similar to Lake View testing).
5. Coordinate with SAI and OSMOS to develop appropriate SERVICE level test protocol.
6. Provide load and deflection instrumentation to augment OSMOS data and provide test control.
7. Report all activities and provide raw and processed data to SAI.
8. Assist in assessing test results as required.

Project B: Evaluate and demonstrate prestressed repair techniques.
*Prior to **Project A***, PITT, working with SAI and PennDOT, will assess the two bridge girders noted while they remain *in situ*. Prototype repairs will be designed and extensive predictions – in support of **Project A** will be conducted. **Project A** testing will only be conducted at SERVICE load levels.

*Following **Project A***, appropriate retrofit measures will be applied using a subcontractor specializing in this application. PITT will conduct the following tasks:

1. Invite OSMOS to return to instrument beams.
2. Duplicate **Project A** SERVICE level test protocol thereby providing a direct 'before and after retrofit' assessment of girder performance.
3. Conduct STRENGTH level tests and eventually tests to failure of retrofit beams to assess their ultimate performance.
4. Contract removal and disposal of beams.

Project B will be documented in a manner suitable for use as training or informational material. Digital video of all stages of repair mobilization and installation and testing will be produced. The test schedule will be announced in advance to permit PennDOT to invite interested parties to attend.

7.2 Proposal to Assess Transfer or 'Redevelopment Length' of Prestressing Strand

An additional small project is proposed to address a question arising from the present research. This project would run concurrently with Project A, above. Its findings would help to inform Project B.

Vehicle impact and/or corrosion can result in the loss of prestressing steel at a girder section. A subsequent analysis of the girder typically removes the broken strands from consideration. However the loss of strand area at one girder section does not imply loss of that strand over the length of the girder.

Two mitigating issues arise:

1. Often the damage is localized. In the Lake View Drive Bridge investigation, for instance, it was noted that the concrete was generally quite sound. Provided the strand was embedded in sound concrete, it was observed to be in excellent shape. There was at least one documented case (Figure 7.1), where a strand was completely lost to corrosion, but only 12 inches along the strand, where it remained embedded in sound concrete, the strand was 'bright steel' with no evidence of deterioration. In such a case, the strand can likely be redeveloped and therefore its area may be accounted for in determining the ultimate capacity of a section at a distance from the damaged region. The strand does not have to have any prestress force remaining – it only needs to be developed – to contribute to the ultimate capacity of the girder.
2. Going a step further, if the strand can be redeveloped, it should still be contributing some degree of prestress force at distances beyond the transfer length of the strand from the damage site.

It is the second point that is investigated in this proposed investigation:

1. *Can the prestress force be redeveloped over the transfer length of the strand?*
2. *If so, to what extent?*
3. *What is the transfer length?*
4. *Finally, does the nature of the damage affect this 'redevelopment'?*

The third question is based on the assumed 'dynamic' or high strain rate strand failure that may be associated with a vehicle impact versus the more gradual (wire-by-wire) failure associated with corrosion. It is anticipated that the former would be the critical case, resulting in a longer redevelopment length.

It is proposed that the PITT team be provided access to decommissioned prestressed girders. While these may be damaged, the proposed experiments would be carried out on relatively undamaged portions of the girder. It is proposed that 'girder 9' from the Lake View Drive Bridge, presently located at the Laboratory Hill – I-79 interchange would be suitable for this proposal. The objectives of the proposed test are:

1. Establish the level of existing stress in the prestressing strand.
2. Establish the apparent 'redevelopment' length associated with strands being ruptured at a relatively high strain rate.

7.2.1 Test Method/Protocol

1. The girder will be shored for both safety (many strands will be cut by the end of the test) and to affect controlled boundary conditions for the test section.

2. A slot will be cut through the cover concrete exposing the strands. The slot will only be large enough to accomplish the necessary instrumentation.
3. One wire of each seven-wire strand will be instrumented with a 2 mm strain gage and an acoustic emission sensor or a PZT.
4. Strands will be cut using a concrete cut-off saw, thereby replicating a relatively high strain rate failure. The use of the acoustic emission sensors should permit 'quantification' of the cutting process (e.g.: the wire-by-wire rupture in the seven-wire strand). This will also permit the investigators to know the order in which the single instrumented wire is ruptured (since seven wire strand is helical, the instrumented wire will rotate around the strand and will be cut in a different sequence based on the distance the cut is made from the gage).
5. The first strand will be cut within the slot. The change in strain in the strand adjacent the cut will be used to establish the existing level of prestress.
6. Subsequent strands will be cut at increasing distances from the slot. Assuming redevelopment of the strand occurs, the apparent strain drop following cutting should also fall. Eventually, it is assumed that a cut beyond the 'redevelopment' length will have no affect on the instrumented strands. Initially, it is proposed that the cuts be made at increments of $0.20\ell_{tr}$, where ℓ_{tr} is the calculated transfer length of the strand.
7. It is hoped that the series will permit at least one duplication of each distance tested.

The process and details are shown schematically in Figure 7.2.

7.2.2 Deliverables

The proposed testing will allow the PITT team to provide a recommendation on the inclusion of ruptured strands in analyses conducted for sections at a distance from known damage. Presently, these strands are discounted completely. Additionally the results will provide data to support anticipated best-practices recommendations on the repair of damage prestressed concrete girders. Specifically, the required repair details ***along*** the girder length. Finally a method for nondestructive assessment of prestress loss can be validated.

Figure 7.1 Strand following induced failure of Lake View Drive girder. The 3/8" strand is entirely lost to corrosion at right of image. At left of image, where it was pulled out of sound concrete (left of red paint line) the strand is 'bright steel' with no section loss. (photo: Harries)

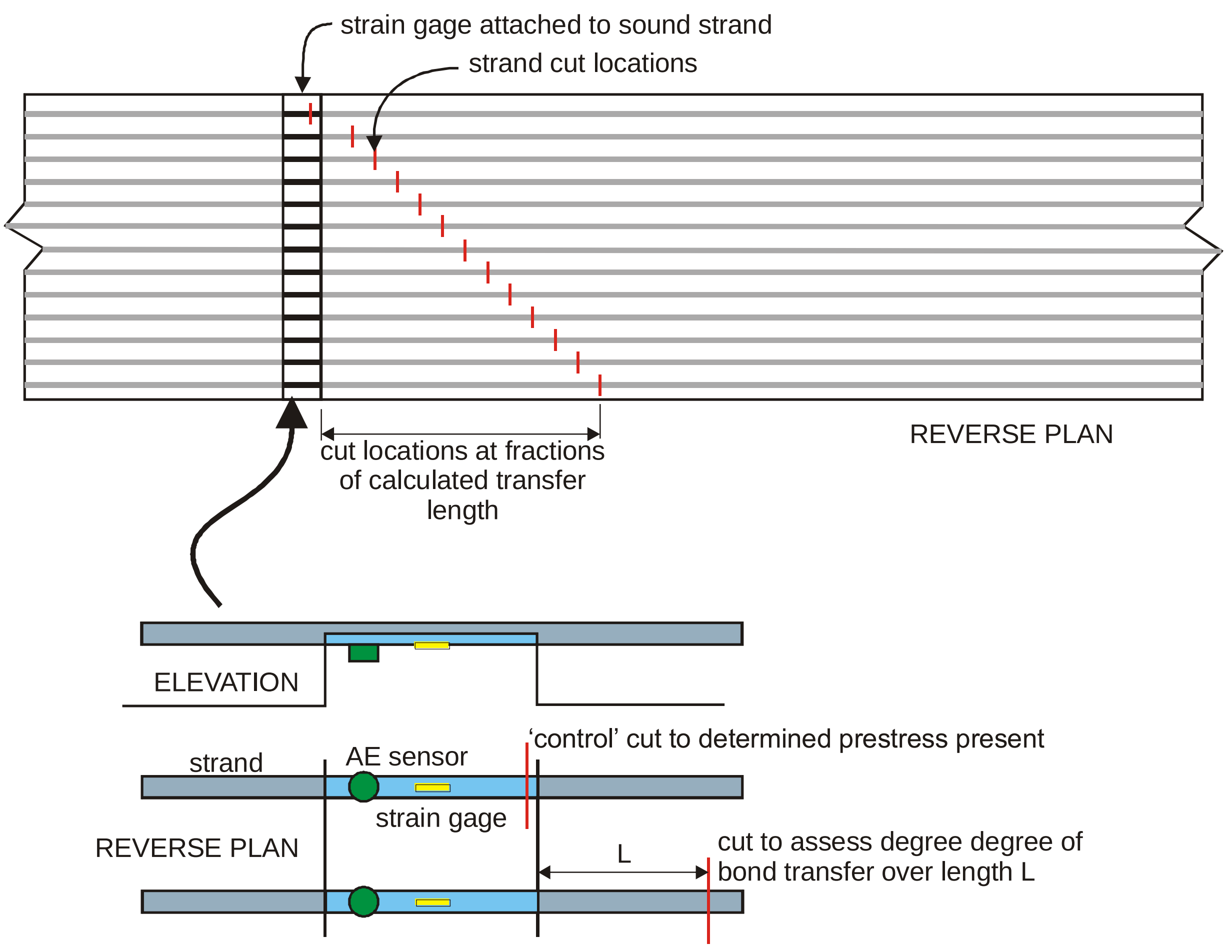

Figure 7.2 Schematic representation of proposed test program.

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APPENDIX A – Notes from Inventory Review (Task 1).

BMS# 02-0376-0130-1385 (Bridge Code A)

- Built in 1962
- In 2004:
    - Heavy erosion at piers near creek
    - Deck Drains clogged with soil
- From Inspection Photos:
    - Span 2, Pier 1 – extensive spalling at mid section
    - Span 2, Pier 1 – visible concrete spalling at pier connection together with strand exposure and heavy corrosion
        - First layer of strands exposed and broken, second layer is exposed and corroding
    - Span 2, Pier 2 – extensive corrosion and spalling at support
    - Many beams have end spalling, exposed strands and stirrups and visible corrosion
    - Beam 12 has apparent impact damage – exposed and broken strands with spalling
    - Beam 12 has significant underside strand exposure (spalling) believed to be from corrosion

Beam 12 – Strand unraveled and 4” hole

Beam 12 – Exposed strands near midspan, apparent impact damage

Beam 11 – exposed strands and reinforcing with heavy corrosion

Beam 12- underside

BMS# 02-0885-0230-0000 (Bridge Code B)

- Full height crack in far abutment – bleeding rust
- Far left wing distress – lateral movement of superstructure – reinforcing exposed and large cracking
- Deck rotation at far left corner
- Possibly scratch marks on beams (14’-4”)
- Beam size S3904, Type B
- NO loss or exposure of prestressed strands
- Girders in relatively good shape

BMS# 02-0885-0221-1126 (Bridge Code C)

- Built in 1963
- From photos, it can be seen that drains have been clogged
- Crushing and/or shear related cracks at beam ends in a few places
- Seems to be impact damage*
  - This damage is on the far side of the bridge considering the direction of traffic
  - Slight spalling and one (possibly more) strand exposed

View from afar

Close up of impact damage

Beam end spall and parapet spalling

Efflorescence and cracking at beam end

BMS# 02-2046-0040-1678 (Bridge Code D)

- 9 beams in an adjacent box arrangement
- Cracking on wearing surface
- Most beams showing some light spalling
- Beams 2, 3 (on left and minimal on right) and 8 - ice hanging in between beams
- Beam 8 - has spalling near handrail
    - Also exposed strand
- Beam 8 – crack in curb and parapet wall over B8
    - Crack in parapet is vertical with efflorescence
- Cracks typically wet or show moisture in inspection photos
- Beam size is 12” x 48”

Beam 8 – Spalling, exposed strands and rust on side

Beam 8 - Close up of broken strands

Spalling and strand exposure at beam end

Continuation of spalling and strand exposure

BMS# 02-2046-0060-0780 (Bridge Code E)

- 7 beams in adjacent box arrangement (dimensions 12” x 36”)
- Beams 6 & 7 at midspan
  - Signs of lateral movement
  - Delamination and spalls
- Centerline post tensioning cable is broken
- 4” of asphalt wearing surface
- At times the asphalt wearing surface had completely worn off and the tops of beams were visible
- A gap between beams 6 and 7 was repaired but the filler material has since fallen out
- Moisture and stains are present
  - Inspection from 2006 said no moisture or staining present so this may be new
- Holes from previous location of the guard rail were not filled in. At this location there is now spalling and visible strands.
  - This spalling may be a consequence of a vehicle impact to the guard rail

Underside of bridge with moisture and staining in between beams

Filler material between Beams 6 & 7

Spalling and exposed strands on exterior beam

Close up of location of former guard rail location

BMS# 04-4020-0030-0657 (Bridge Code F)

- Deck in relatively poor condition
    - Cracks
    - Holes
    - Top of Pier 3 shows accidental buildup of asphalt meaning a hole in the deck
- Left parapet wall spalling
- Pier 1 cap – severe and significant underside spalling
- Diaphragm @ Piear 3 – cracking and spalling
- Beam 5, Span 1 @ N. Abutment – beam end has spalling and exposed strands with rust

Hole in joint over pier

Hole in FLT paved joint

BMS# 04-4035-0040-0525 (Bridge Code G)

- Deck in poor condition – many crack and holes (especially at joints)
- Piers have significant spalling
- Diaphragms between beams show significant spalling and reinforcement rust
- Severe spalling on underside of parapet wall near Pier 3
    - Water staining and efflorescence underneath
- Beam 9, Span 1, on Pier 1 – diagonal cracking that may be shear related
- In 1996: some cracking and rust stains at beam ends over piers
- 2000: Delamination at some beam ends with spalling and strand exposure (no good photos in inspection report)
- 2000: Beam ends at Piers have heavy cracking in 45^{o} x's (no good photos in inspection report)

Far approach relief joint and approach slab

Hole through joint at Pier 3

Sever spalling and reinforcement exposure on underside of parapet wall at Pier 3

Cracking at the end of Beam 9, Span 1

BMS# 02-3051-0050-0295 (Bridge Code H)

- Spans 1 and 3 are spaced box beams and Span 2 is adjacent box beams
- All downspouts and inlets in poor working order
- Heavy cracking in Deck
- Rebar in Deck is exposed near Pier 1
- Large Longitudinal crack in right approach shoulder in Span 1
- Spalling with exposed rebar in underside of Deck between beams 4 and 5 at near abutment. Possible water damage
- Beam 1, Span 2 – post tensioning anchor is broken
- Beam 9 and 10 at Pier 2 – rust stains
- Span 2 – heavy leakage between beams

Heavy leakage between beams in Span 2

Exposed rebar in the underside of the deck between beams 4 and 5

Clogged inlet of Span 1

Poor downspout condition, typical for all on this bridge

BMS# 02-0279-0144-0000 (Bridge Code J)

- Longitudinal crack on B1’s bottom flange
- Minimum of 15’-0” vertical clearance
- Remaining superstructure in good shape

Longitudinal cracks in the bottom flange of B1

Longitudinal cracks in the bottom flange of B1

Longitudinal cracks in the bottom flange of B1

Spalling under B1 at NAB

BMS# 02-3084-0020-1033 (Bridge Code K)

- Hole in joint at Pier 2
- Span 2 pothole with exposed rebar
- Span 3, Beam 8 – collision damage with exposed and severed strands
- Span 3, Beam 6 – collision damage with spalling
- Damage located at 17’ from Rt abutment, about 3’ long
- Inspection report indicates that vertical clearance decreased from 14’-10” to 14’-6” in 1994

Span 3, Beam 8 – collision damage with exposed and severed strands

Span 3, Beam 8 – close up

Location of collision damage on Beam 8

Span 3, Beam 6 – collision damage with spalling

BMS# 02-4022-0020-0000 (M)

- Issues with: deck, railings, piers, differential settlement, but no pictures of impact damage
- Last inspection 8/07
- Vertical clearance left and right is 15'-11" and 16'-4", respectively

*No pertinent images

BMS# 37-4012-0010-0698 (Bridge Code N)

- Many holes and patching in wearing surface/deck
- Holes in joints
- Efflorescence, rust and some cracking on piers
- Impact damage on Beam 1, Span 2

Collision damage on Beam 1, Span 2

4 broken strands in Beam 1 at location of collision damage

BMS# 04-0060-0160-1497 (Bridge Code P)

- Spalling and exposed strands on corners on interior beams
- Scraping on underside of beams
- Collision damage
- Significant longitudinal cracking

Collision damage

Longitudinal crack from impact damage

Scraping on underside of Span 4

Bottom corner of beam S7, cracked and delaminated at pier

BMS# 04-3017-0020-0754 was a bridge on the 'hit list' but is not discussed here. Since this bridge was damaged by impact, it has been replaced; therefore warranting no further evaluation for this project.

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APPENDIX B – Survey Instrument and Cover Letter (Task 4).

University of Pittsburgh

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412-624-9870
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January 3, 2008 ***electronic transmission via PennDOT***

THIS SURVEY IS DISTRIBUTED TO STATE/PROVINCIAL/JURSIDICTIONAL BRIDGE ENGINEERS. IF IT IS MORE APPROPRIATE IN YOUR JURISDICTION FOR THE MAINTENEANCE ENGINEERS TO RESPOND PLEASE FORWARD APPROPRIATELY

RE: REPAIR METHODS FOR PRESTRESSED CONCRETE BRIDGES – STATE OF PRACTICE SURVEY

The Structural Engineering and Mechanics Group in the Department of Civil Engineering at the University of Pittsburgh, funded by the Pennsylvania Department of Transportation (PennDOT), is conducting a study aimed at establishing the state of practice for the repair of structurally damaged prestressed bridge elements.

Your assistance is requested in completing the attached survey. The objective of the survey is to establish the state of practice for the repair of structurally damaged prestressed bridge elements. This survey has been approved by PennDOT and the University of Pittsburgh Institutional Review Board and is being sent to all US State DOTs, Canadian Provincial MOTs and some other identified users.

Specifically, the survey respondent is asked to *please consider only prestressed or post-tensioned concrete bridge structures damaged once they have been placed in service in your jurisdiction. Do not consider fabrication, shipping or non-structural erection-induced damage to precast concrete elements*.

The survey responses will be tabulated and all identifying remarks stricken prior to any publication of results. In this way, presentation of the responses will be anonymous to all but myself, the graduate student assisting with this project and the project oversight committee. Nonetheless, we ask that you provide your contact information so that we may “check off” your organization’s response and provide you a copy of the survey results.

We ask that you complete the survey, preferably electronically (the survey is provided in MSWord and Adobe PDF format for your convenience), and return it before **JANUARY 31, 2008** to:

Dr. Kent A. Harries, P.Eng.
kharries@engr.pitt.edu
fax: 412.624.0135

Thank you for your assistance with this survey. Please feel free to contact me at any time.

Sincerely,

Kent A. Harries, Ph.D., P.Eng.
William Kepler Whiteford Faculty Fellow
Assistant Professor

State-of-Practice Survey
Repair Methods for Prestressed Concrete Bridges

Survey completed by:

Name:	______________	Title:	______________
Jurisdiction:	______________	email:	______________
Address:	______________	Telephone:	______________
	______________		

PLEASE RETURN COMPLETED SURVEY BEFORE JANUARY 18 TO:

Kent A. Harries
University of Pittsburgh
Civil and Environmental Engineering
936 Benedum Hall
Pittsburgh PA 15261
fax: (412) 624-0135
kharries@engr.pitt.edu

Introduction

In responding to this survey, please consider only prestressed or post-tensioned concrete bridge structures damaged once they have been placed in service in your jurisdiction. Do not consider fabrication, shipping or non-structural erection-induced damage to precast concrete elements.

The purpose of this research study is to assess the current state of practice associated with the structural repair of prestressed concrete bridge elements. This survey is being distributed to all PennDOT district offices, US states DOTs, Canadian MOTs and other jurisdictions having responsibility for prestressed concrete bridges (such as transit authorities, US Forest Service, etc.). If you are willing to participate, you will be asked to provide your professional contact information and direct responses to the survey questioned asked. There are no foreseeable risks associated with this project, nor are there any direct benefits to you. Your participation is voluntary. All surveys will be kept in confidence and responses will be stripped of remarks identifying an individual, organization or jurisdiction. This study is being conducted by Dr. Kent A. Harries, who can be reached at 412.624.9873 of kharries@engr.pitt.edu, if you have any questions.

1. Plans and specifications for repair of damaged prestressed girders are usually prepared by (check all that apply):

- [ ] DOT design/bridge engineer
- [ ] DOT maintenance engineer
- [ ] Private consultant
- [ ] Standard details/specifications used
- [ ] Other (please describe) ______________________________

2. Construction of repair is usually carried out by (check all that apply):

- [ ] DOT/agency personnel
- [ ] Private contractor
- [ ] Other (please describe) ______________________________

3. Rate the following factors by importance in the determination of the method or repair.

	low	**moderate**	**high**	**not considered**
Cost of repair				
Time required to make repair				
Aesthetics of repair				
Interruption of service				
Load capacity				
Expected service life of repair				
Maintenance required				
Other, please specify:				

____

4. Estimate the number of prestressed concrete bridges damaged in your jurisdiction over the last five years for the following degrees of damage:

		check here if your jurisdiction does not track this level of damage
	Minor damage concrete cracks and nicks; shallow spalls and scrapes not affecting tendons	
	Moderate damage large concrete cracks and spalls; exposed, undamaged tendons	
	Significant damage exposed and damaged tendons; loss of portion of cross section	
	Severe damage damage severe enough to result in girder distortion or misalignment	

5. What actions are typically taken for the following degrees of damage (use damage definitions from Question 4)**?**

	no repair made	**non-structural repair**	**load-carrying repair**	**replace member or structure**
Minor damage				
Moderate damage				
Significant damage				
Severe damage				

6. What are the typical causes of damage to prestressed concrete members <u>that require repair action to be taken</u> (check all that apply)**:**

common	**rare**	
		Vehicle impact
		Corrosion occurring at unrepaired site of vehicle impact
		Corrosion resulting from source **other than** vehicle impact
		Natural hazard (earthquake, hurricane/tornado, etc.)
		Construction error (misplaced reinforcing steel, low strength concrete, etc.)
		Nonspecific deterioration due to aging
		Other (please describe)

7<u>. In cases where repair action is eventually taken</u>, what procedures are used to determine the extent of the damage?

commonly	**rarely**	
		Visual inspection only
		Non destructive evaluation (NDE/NDT); which methods are typically used? ______
		Destructive evaluation; which destructive methods are typically used? ______
		Other (please describe) ______

8. Briefly describe what analytical procedures are used to assess the damage and the need for repair of prestressed concrete elements (include names of software if appropriate)

9. Methods of repair of MINOR damage (concrete cracks and nicks; shallow spalls and scrapes not affecting tendons) used are (check all that apply):

common	rare	
		Do nothing
		Repaint surface
		Patch concrete
		Other (please describe)

10. Methods of repair of MODERATE damage (large concrete cracks and spalls; exposed, undamaged tendons) used are (check all that apply):

common	rare	
		Do nothing
		Patch concrete
		Epoxy injection
		Concrete removal/surface preparation prior to patching
		Clean tendons
		Installation of active or passive corrosion control measures
		Other (please describe)

11. Methods of repair of SIGNIFICANT damage (exposed and damaged tendons; loss of portion of cross section) used are (check all that apply):

common	rare	**Repair of tendons**
		Cut tendons flush with damaged section (no tendon repair)
		External post-tensioning
		Internal splices (describe below) …is repair re-stressed? yes ☐ no ☐
		Metal sleeve slice …is repair re-stressed? yes ☐ no ☐
		Combination splice (describe below) …is repair re-stressed? yes ☐ no ☐
		Externally applied reinforcing material (FRP, etc.) (describe below)
		Installation of active or passive corrosion control measures
		Other (please describe)

common	rare	**Repair of concrete**
		Concrete removal/surface preparation prior to patching
		Patch concrete
		Epoxy injection
		Other (please describe)

common	rare	
		Replace individual girder
		Replace bridge

Please describe methods used (Question 11)

12. Methods of repair of SEVERE damage (damage severe enough to result in girder distortion or misalignment) used are (check all that apply):

common	rare	
		Repair of distortion/misalignment
		External post-tensioning
		Jacking and re-use of damaged member
		Jacking and replacement of damaged member
		Provision of new/additional permanent supports (extended corbels, etc.)
		Other (please describe)

____

____

common	rare				
		Repair of tendons			
		Cut tendons flush with damaged section (no tendon repair)			
		External post-tensioning			
		Internal splices (describe below)	…is repair re-stressed?	yes ☐	no ☐
		Metal sleeve slice	…is repair re-stressed?	yes ☐	no ☐
		Combination splice (describe below)	…is repair re-stressed?	yes ☐	no ☐
		Externally applied reinforcing material (FRP, etc.) (describe below)			
		Installation of active or passive corrosion control measures			
		Other (please describe)			

____

____

common	rare	
		Repair of concrete
		Concrete removal/surface preparation prior to patching
		Patch concrete
		Epoxy injection
		Other (please describe)

____

____

common	rare	
		Replace individual girder
		Replace bridge

Please describe methods used (Question 12)

13. How well do your prescribed repair methods perform? Please identify problems and/or successes experienced in the repair of damaged prestressed concrete bridge elements in your jurisdiction.

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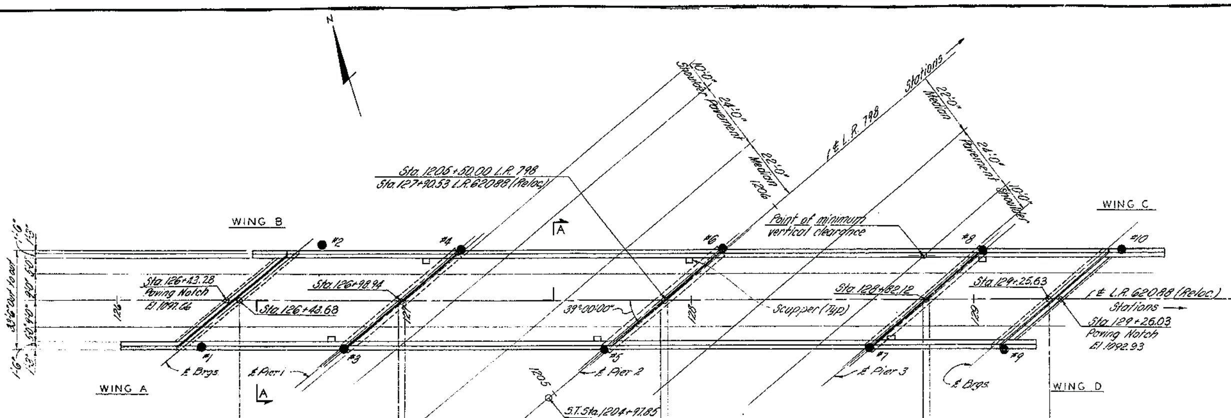
APPENDIX C – Representative Structure Drawings (Task 5).

C.1 Bridge LV

Structural drawings for bridge LV (Spancrete 1960).

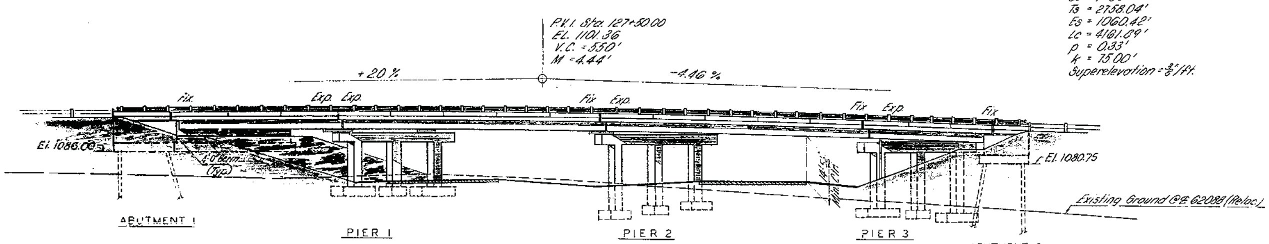


● —Indicates core boring location

PLAN
Scale 1" = 20'

CURVE DATA L.R. 798

P.I. Sta. 1187+94.00
Δ = 86°-14'-16" Lf.
D = 2°00'
Rc = 2864.93
Ls = 150.00'
θs = 1°30'
Ts = 2758.04'
Es = 1060.42'
Lc = 4161.89'
p = 0.33'
k = 75.00'
Superelevation = 3/8"/Ft.



ELEVATION
Scale 1" = 20'

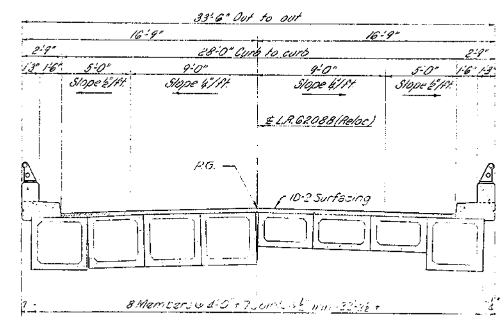
NOTES:

1 — Live load: H 20-S16-44.
2 — Design:- A.A.S.H.O. Standard Specifications for Highway Bridges (1953)
Earth pressure: 120 lbs./cu.ft. vertical; 35 lbs./cu.ft. fluid pressure horizontal

3 — Specifications: — Penna. Department of Highways Forms 408 (1954) and 409 (1949).
4 — All superstructure concrete shall be Class A. All abutment concrete shall be Class B except wingwall parapets which shall be Class A. All concrete in piers shall be Class A except footings which shall be Class B.
4a. Embankment shall be thoroughly compacted up to bottom elevation of Abutment before driving piles.
5 — All exposed concrete edges shall have a 1"×1" chamfer unless noted.
6 — Reinforcement bars are designed for fs = 18,000 psi & are detailed in accordance with A.C.I. recommendations.
7 — All reinforcement bars carrying design or temperature stresses shall be lapped at least 40 diameters where splicing is required.
8 — The bases of the structure may be ordered by the Engineer to be at any elevation or of any dimensions necessary to provide a proper foundation.
9 — Place two-coat painted waterproofing on top of footing heels and on rear face of abutment walls and wingwalls up to a line 1'-0" below top of finished grade.
10 — Fabricated structural steel shall be A.S.T.M. designation A7, current
11 Provide 2" Cover on reinforcement bars unless otherwise Noted.

12 — W.P. - denotes Work Point.
13 — For parapet railing and guard fence connections see P.D.H. Std. Dwgs. [illegible] S-3361.
14. — Prestressed, Precast beams shall be designed in accordance with Pennsylvania Department of Highways Specifications Form 408 (1954) Supplement Section 6.24 (1958 [illegible]

15. — All piles shall be steel beam piles 10 BP42.
16. — Piles shall be driven to refusal or to practical refusal as directed by the Engineer but, in no case, to a bearing value less than 72 Tons for Test Piles and 60 Tons for 10 BP42.
17. — All dowels in Pier Cap shall be set in preformed holes. Drilling of holes for dowels in Pier Cap will not be permitted.
18. — All Parapet, Parapet Railing will be considered a part of the Bridge Superstructure.

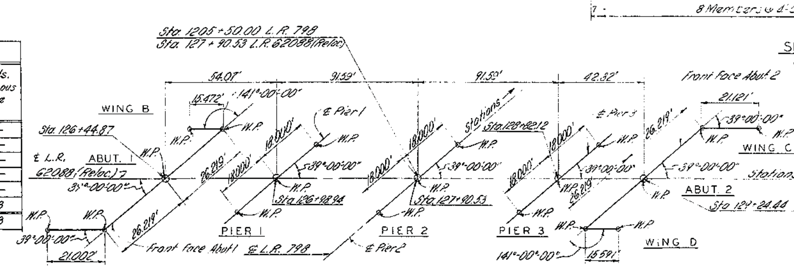


SECTION A–A
Scale 1/4" = 1'-0"

QUANTITIES

Item	Cu. Yds. Class 3 Excavation	Cu. Yds. Class A Concrete	Sq. Ft. Prestressed Precast Conc. Box Beam	Cu. Yds. Class B Concrete	Lbs. Reinforcement Bars	Lbs. Fabricated Structural Steel	Lin. Ft. Parapet Railing	Steel Beam Piles 10 BP 42: Test Piles	Steel Beam Piles 10 BP 42: Lin. Ft.	Each Guard Fence Connections	Sq. Yds. Bituminous Surface 10-2
Abutment 1	—	4	—	104	5,530	—	27	1 @ 20'	300	2	—
Pier 1	98	46	—	25	13,840	—	—	—	—	—	—
Pier 2	89	49	—	18	17,440	—	—	—	—	—	—
Pier 3	74	53	—	25	15,310	—	—	—	—	—	—
Abutment 2	—	4	—	108	5,590	—	27	1 @ 32'	470	2	—
Superstructure	—	114	9,143	—	8,770	6450	570	—	—	—	878
Totals	261	270	9,143	280	66,480	* 6450	624	Lump Sum	770	4	878

* Includes: 3720 Lbs. Malleable iron or Cast Steel
2340 Lbs Wrought iron



STAKE-OUT PLAN
Not to Scale

These drawings supersede S-3661 Approved June 16, 1959

PRESTRESS ALTERNATE REVISIONS BY
MACOMBER & FABER
CONSULTING ENGINEERS
HARRISBURG, PENNA.

PREPARED BY
GROF & MYERS
CONSULTING ENGINEERS
HARRISBURG, PENNA

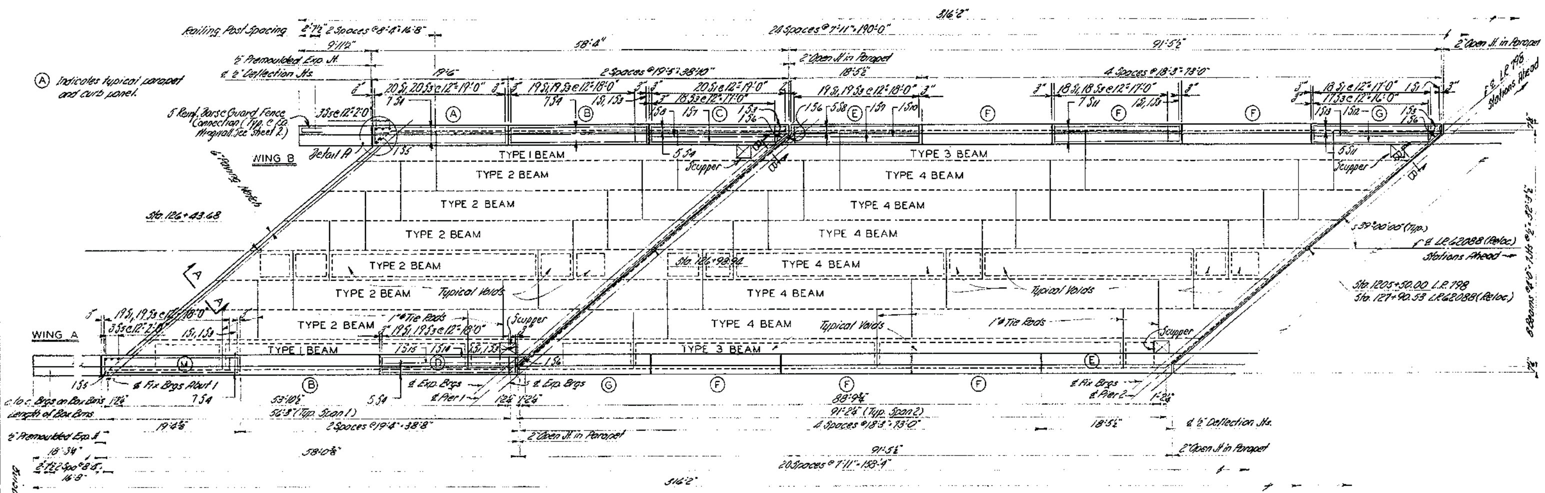
APPROVED JUN 13 1960
BRIDGE ENGINEER

Commonwealth of Pennsylvania
Department of Highways
BRIDGE UNIT

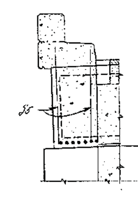
WASHINGTON COUNTY
L.R. 798 SECTION 1
L.R. 798 UNDER L.R. 62088 (RELOC)
GENERAL PLAN
STA 1205+50.00

SCALE: AS NOTED
SHEET 1 OF 6 + S-1614 B, S-3361
S-3661 A

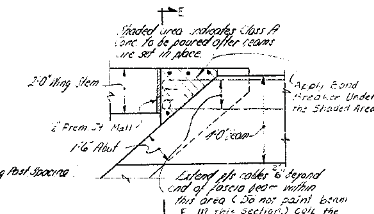
DESIGNED F.S.F.
DRAWN W.W.W.
TRACED W.W.W.
CHECKED E.R.L.



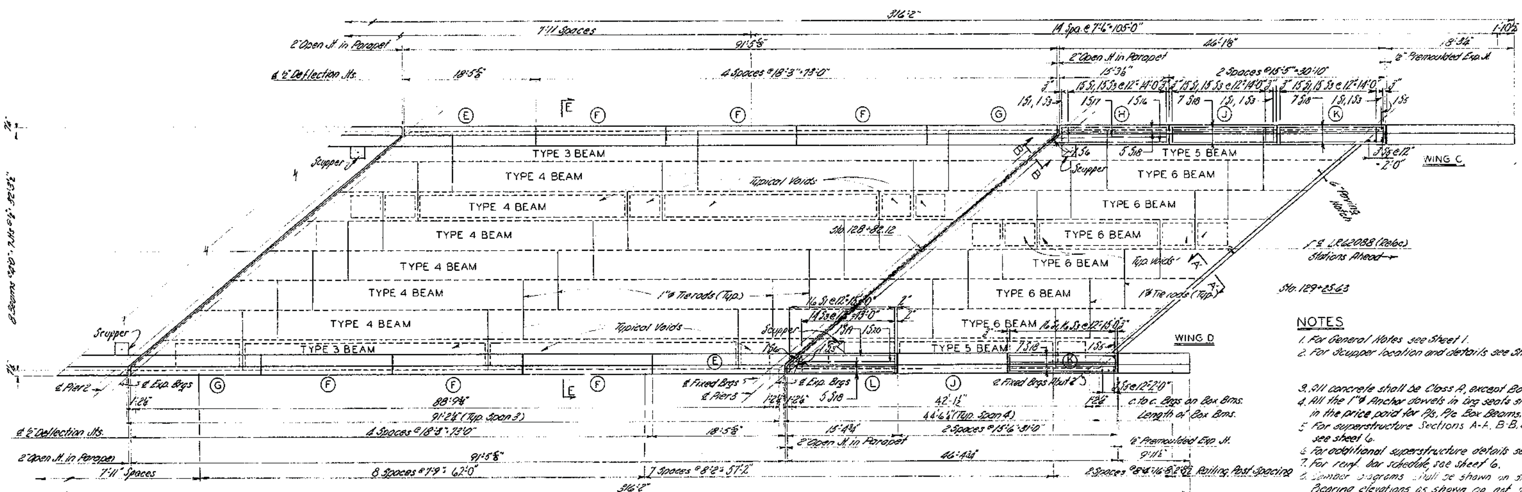
PLAN - SPANS 1 & 2
Scale: ⅛"=1'-0"



SECTION E-E
Scale: ⅜"=1'-0"



DETAIL A
Scale: ⅜"=1'-0" (Typical Detail at Abut.)



PLAN - SPANS 3 & 4
Scale: ⅛"=1'-0"

NOTES
1. For General Notes see Sheet 1.
2. For Scupper location and details see Sheet 7.

3. All concrete shall be Class A, except Box Beams.
4. All the 1"⌀ Anchor dowels in brg. seats shall be included in the price paid for P/s, P/c Box Beams.
5. For superstructure Sections A-A, B-B, and E-E see sheet 6.
6. For additional superstructure details see sheet 6.
7. For reinf. bar schedule, see sheet 6.
8. Camber diagrams shall be shown on shop drawings. Bearing elevations as shown do not provide for beam camber. The beam camber shall be checked by the contractor. If the beam camber exceeds the vertical curve ordinate for its span, the bearing elevations or slab thickness shall be adjusted accordingly at no additional cost to P.D.H.

NO	REVISION	BY	CH'KD	DATE
1	Revised Detail 'A' To Clarify.	H.H.U.	H.E.L.	8-15-60

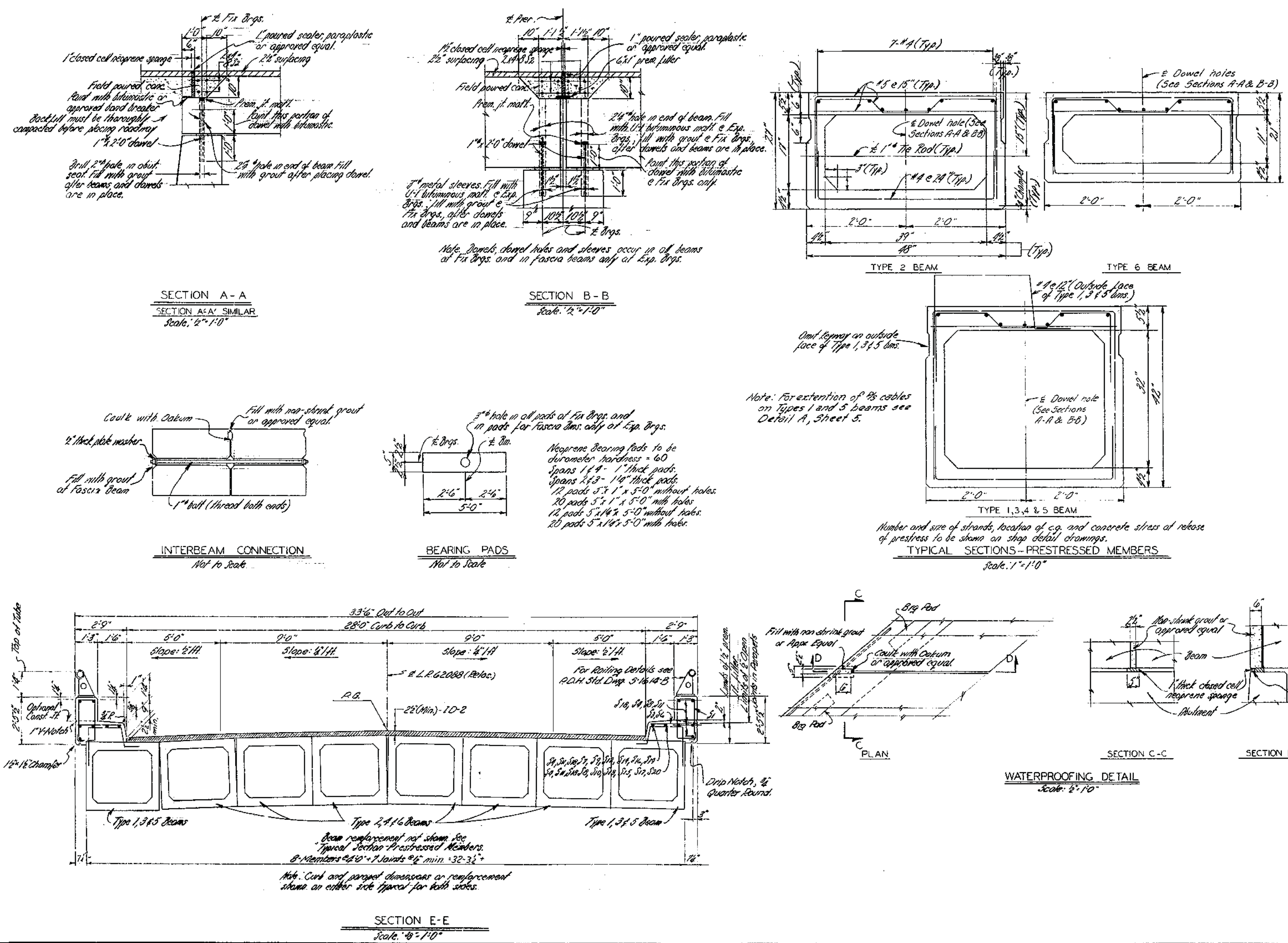
Approved: JAN 13 1960
K.H. Jensen
Bridge Engineer

Commonwealth of Pennsylvania
Department of Highways
BRIDGE UNIT

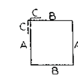
WASHINGTON COUNTY
L.R. 798 SECTION 1
L.R. 798 UNDER L.R. 62088 (RELOC)
SLAB PLAN
STA 1205+50.00

SCALE AS NOTED
SHEET 5 OF 8

S-3661-A



REINF. BAR SCHEDULE									
MK	NUMBER	SIZE	LENGTH	TYPE	A	B	C	D	REMARKS
S1	596	4	6'-3"	7	1'-11½"	11"	3"		
S2	64	6	27'-3"	Str.					
S3	590	4	2'-6"	Str.					
S4	38	4	19'-0"	Str.					
S5	16	4	3'-9"	Str.					
S6	12	4	2'-0"	Str.					
S7	1	4	17'-3"	Str.					
S8	21	4	18'-0"	Str.					
S9	4	4	19'-6"	Str.					
S10	4	4	18'-9"	Str.					
S11	104	4	17'-9"	Str.					
S12	4	4	16'-0"	Str.					
S13	4	4	16'-9"	Str.					
S14	1	4	20'-6"	Str.					
S15	1	4	19'-9"	Str.					
S16	1	4	16'-6"	Str.					
S17	1	4	15'-9"	Str.					
S18	38	4	15'-0"	Str.					
S19	1	4	13'-3"	Str.					
S20	1	4	14'-0"	Str.					



TYPE 7

FOR NOTES SEE SHEETS 1 & 5.

Approved: JUN 13 1960
K H Jensen
Bridge Engineer

Commonwealth of Pennsylvania
Department of Highways
BRIDGE UNIT

WASHINGTON COUNTY

L.R.798 SECTION 1

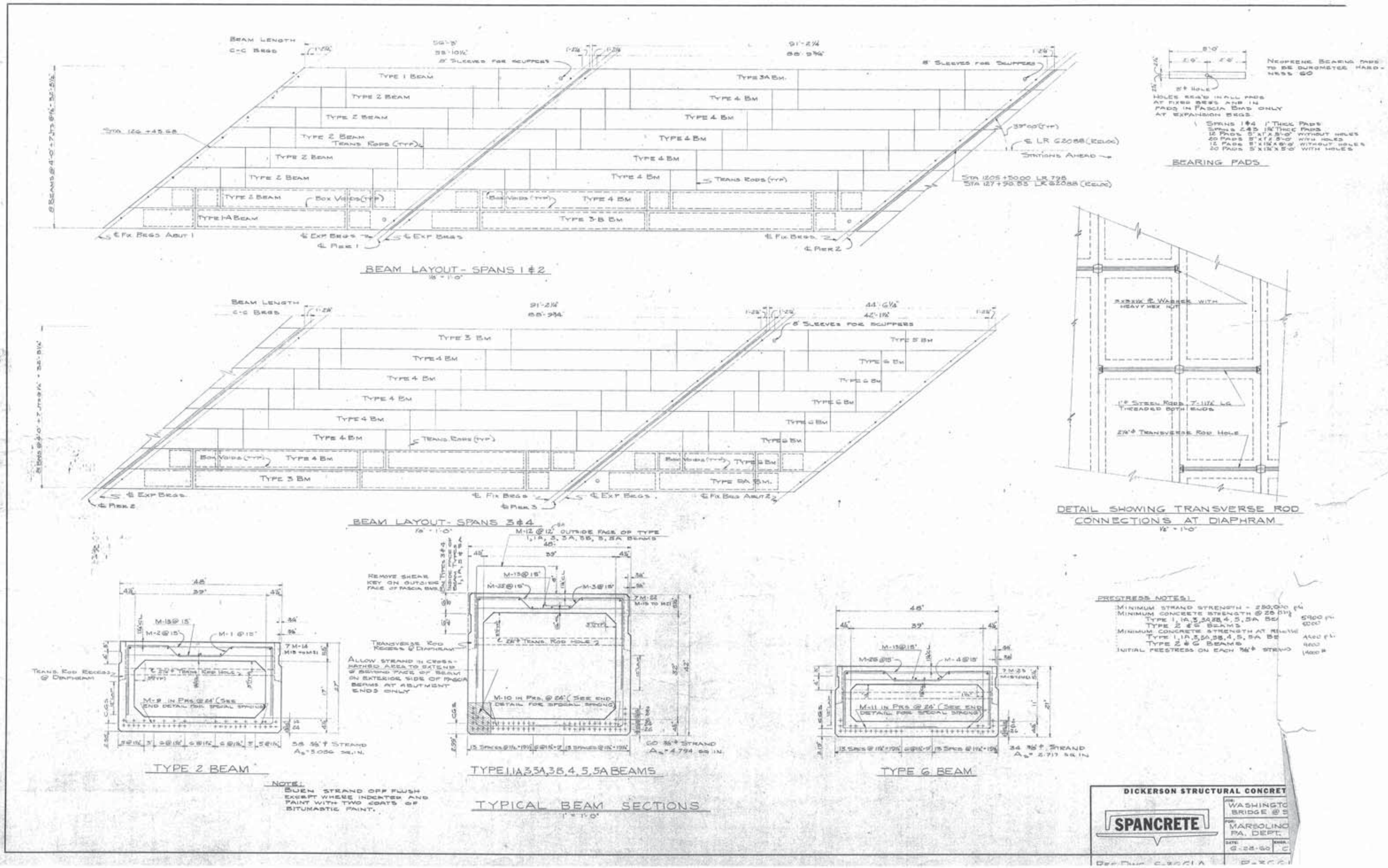
L.R.798 UNDER L.R.62088 (RELOC)

SUPERSTRUCTURE DETAILS

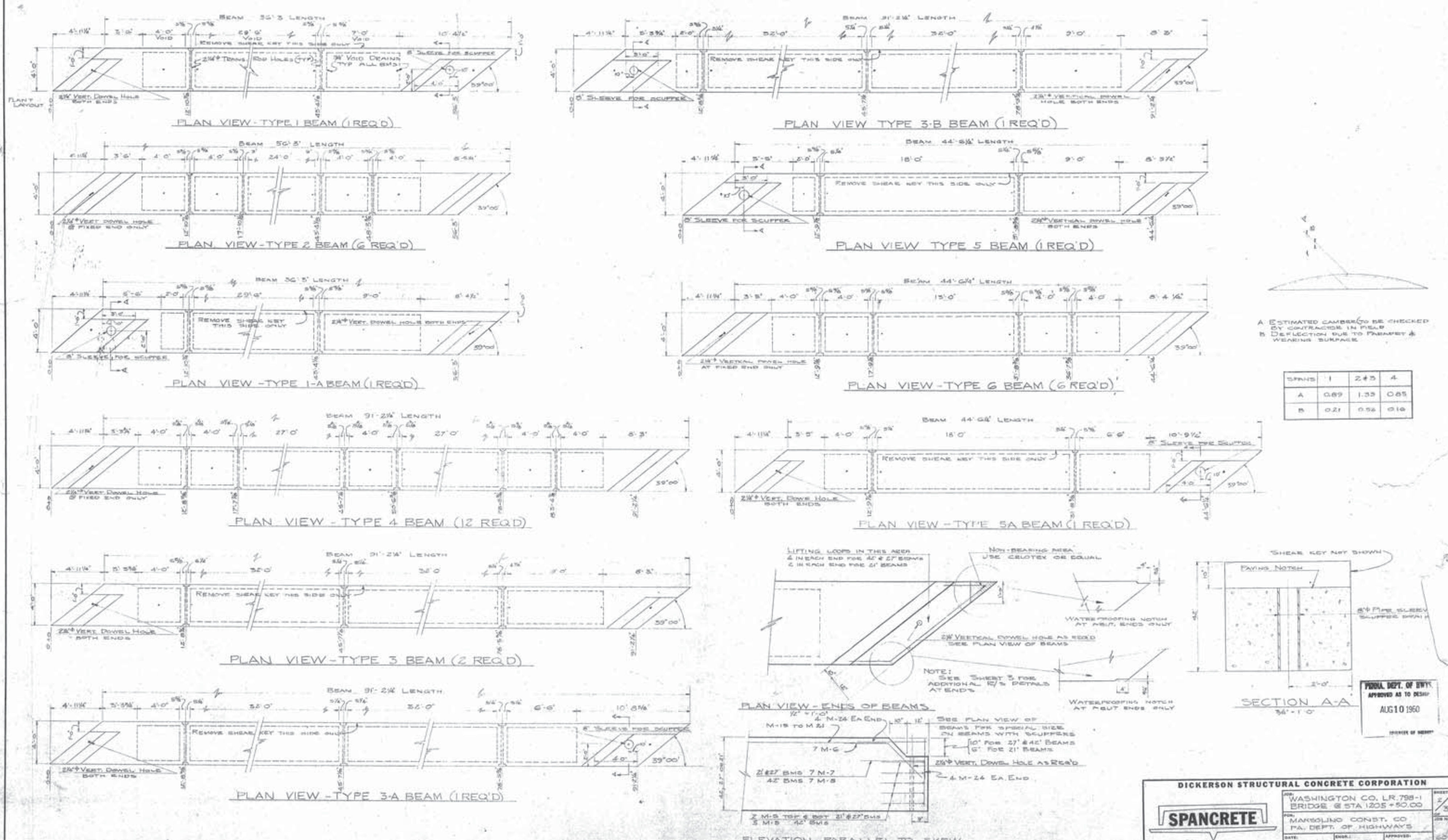
STA.1205+50.00

SCALE: AS NOTED
SHEET 6 OF 8

S-3661-A



BEAM LAYOUT - SPANS 1 & 2
BEAM LAYOUT - SPANS 3 & 4
BEARING PADS
NEOPRENE BEARING PADS TO BE DUROMETER HARDNESS 60
DETAIL SHOWING TRANSVERSE ROD CONNECTIONS AT DIAPHRAM
PRESTRESS NOTES:
TYPE 2 BEAM
TYPE 1,1A,3,3A,3B,4,5,5A BEAMS
TYPE 6 BEAM
TYPICAL BEAM SECTIONS
NOTE: BURN STRAND OFF FLUSH EXCEPT WHERE INDICATED AND PAINT WITH TWO COATS OF BITUMASTIC PAINT.
DICKERSON STRUCTURAL CONCRET
SPANCRETE



SPANS	1	2 & 3	4
A	0.89	1.33	0.85
B	0.21	0.56	0.19

PENNA. DEPT. OF HWYS. APPROVED AS TO DESIGN AUG 10 1960

DICKERSON STRUCTURAL CONCRETE CORPORATION

SPANCRETE

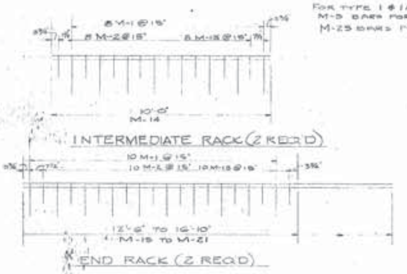
JOB: WASHINGTON CO. LR. 798-1 BRIDGE @ STA 1205 + 50.00

FOR: MARSOLINO CONST. CO PA. DEPT. OF HIGHWAYS

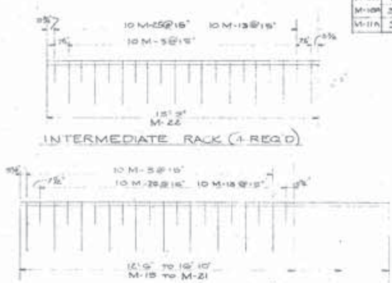
DATE: 7-1-60 | ENGR.: C.G.B. | APPROVED: | SHEET 2 OF 3 | JOB NO. 2014

REF. DWG. 5-3661 A | P-3661

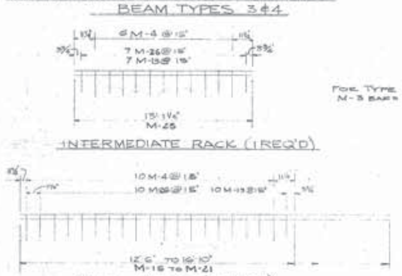
FOR TYPE 1 & 1A BMS. SUBSTITUTE M-3 BARS FOR M-1 BARS & M-25 BARS FOR M-2 BARS



REINFORCING RACK LAYOUT
BEAM TYPES 1 & 2

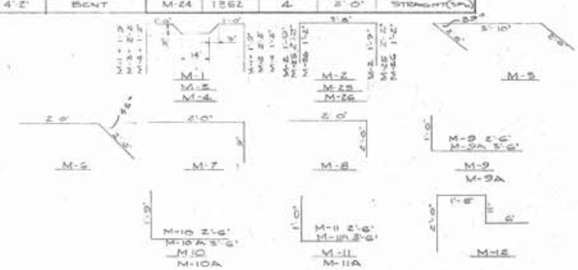


REINFORCING RACK LAYOUT
BEAM TYPES 3 & 4



REINFORCING RACK LAYOUT
BEAM TYPES 5 & 6

BAR LIST				
MARK	NUMBER	SIZE	LENGTH	DESCRIPTION
M-1	216	5	7'-4"	BENT
M-2	216	5	7'-2"	"
M-3	1086	5	8'-2"	"
M-4	180	5	6'-2"	"
M-5	224	6	9'-10"	"
M-6	448	6	4'-0"	"
M-7	224	6	2'-9"	"
M-8	224	6	4'-0"	"
M-9	300	4	3'-6"	"
M-10	1576	4	4'-8"	"
M-11	234	4	3'-2"	"
M-12	320	4	5'-1"	BENT
M-13	1464	5	5'-9"	STRAIGHT
M-14	112	4	10'-0"	"
M-15	64	4	12'-6"	"
M-16	64	4	13'-7"	"
M-17	64	4	13'-11"	"
M-18	64	4	14'-8"	"
M-19	64	4	15'-5"	"
M-20	64	4	16'-9"	"
M-21	64	4	16'-10"	"
M-22	448	4	15'-9"	"
M-23	56	4	8'-1½"	
M-24	1362	4	2'-0"	STRAIGHT (SPL)
M-25	1064	5	8'-2"	BENT
M-26	162	5	6'-0"	"
M-9A	960	4	4'-6"	"
M-10A	3200	4	5'-5"	"
M-11A	960	4	4'-2"	BENT

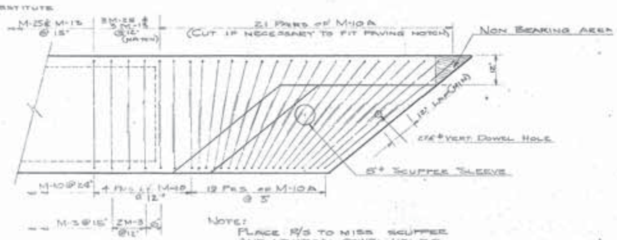


NOTE:
CUT STEEL TO FIT IN BEAMS WITH LARGE PAVING NOTCHES

LIST OF MATERIALS		
NUMBER	UNITS	DESCRIPTION
118,200	L.F.	3/8"ϕ PRESTRESSING STRAND
24	Pcs.	39 X 17" BOX VOIDS 4'-0" LG
6	Pcs	39" X 17" " " 24'-0" LG
68	Pcs	39" X 32" " " 4'-0" LG
8	Pcs	39" X 32" " " 2'-0" LG
1	Pcs	39 X 32 " " 7'-0" LG
8	Pcs	39 X 32" " " 9'-0" LG
2	Pcs	39" X 32" " " 29'-6" LG
27	Pcs	39 X 32" " " 18'-0" LG
2	Pcs	39" X 32" " " 6'-6" LG
24	Pcs	39" X 32" " " 27'-0" LG
8	Pcs	39 X 32" " " 32'-0" LG
24	Pcs	39" X 11" " " 4'-0" LG
6	Pcs	39 X 11 BOX VOIDS 13'-0" LG
712	L.F.	2¼"ϕ TYPE 'A' SONOVOID
70	Pcs	1"ϕ TRANSVERSE RODS 7'-11½" LG
140	Pcs	3X3X½" ℞ WASHERS
140	Pcs	1" HEAVY HEX NUTS
190	Pcs	½" FASCIA FORMING INSERTS
36	SQ. FT	½" THICK CELOTEX OR EQUAL

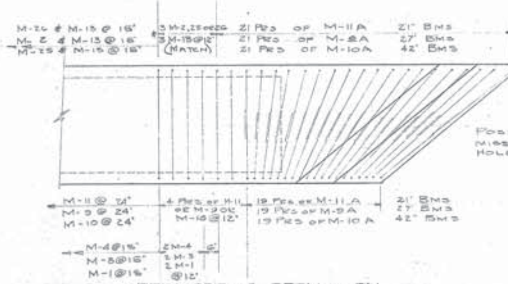
BEAM CONCRETE QUANTITIES AND WEIGHTS		
BEAM TYPE	CONCRETE PER BEAM (CU YDS)	SHIPPING WEIGHT (TONS)
1 & 1A	16.1	33.8
2	11.9	24.9
3	22.6	47.4
3A & 3B	23.4	49.0
4	23.9	50.0
5 & 5A	12.4	25.9
6	8.2	17.2

FOR TYPE 5 & 5A BMS. SUBSTITUTE M-3 BARS FOR M-4 BARS



NOTE:
PLACE R/S TO MISS SCUPPER AND VERTICAL DOWEL HOLES

REINFORCING DETAILS ON
BEAM ENDS WITH SCUPPERS
½" = 1'-0"



POSITION AS REQ'D TO MISS LIFTING INSERTS & DOWEL HOLES

PENNA. DEPT. OF HWYS.
APPROVED AS TO DESIGN
AUG 10 1960
ENGINEER OF BRIDGES

REINFORCING DETAILS ON
BEAM ENDS WITHOUT SCUPPERS

DICKERSON STRUCTURAL CONCRETE CORPORATION

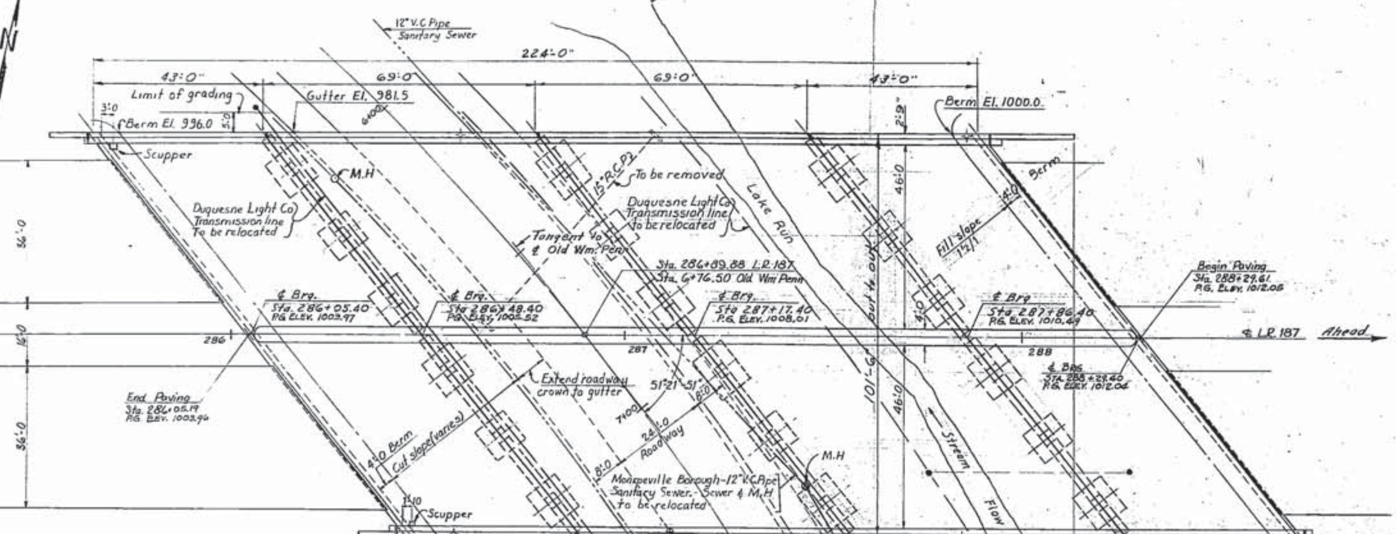
SPANCRETE

WASHINGTON CO. LR 798-1
BRIDGE @ STA 1205+80.00
MARSOLINO CONST. CO.
PA. DEPT. OF HIGHWAYS
SHEET 3/3
JOB NO. 6014
DATE: 7-5-60
ENGR.: C.G.B.
APPROVED:

REF. DWG. S-3661A
P-3661

C.2 Bridge A

Structural drawings for bridge A (PADoH 1960c).

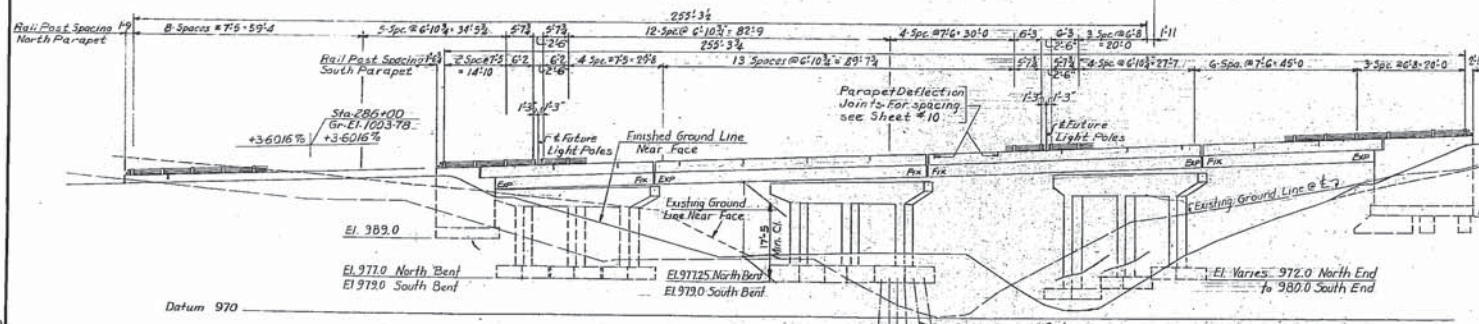


PLAN

Scale: 1" = 15'-0

Horizontal Curve Data Old Wm. Penn

P.I. Sta. 7+19.69
Δ 34°-06'-20"
D - 8°
R - 716.20
T - 219.69
L - 426.29
E - 32.93



ELEVATION

Scale: 1" = 15'-0

Vertical Curve Data L.R. 187

P.I. Sta. 262+50
Elev. 919.14
V.C. 900'
M.O. 9.59'

-4.920%   +3.6016%

Horizontal Curve Data L.R. 187

P.I. Sta. 318+12.21
Δ 55° 49' 35" Rt.
D 1° 30' 00"
R 3919.72'
T 2023.56'
L 3721.76'
S.E. ¼" Per ft.

GENERAL NOTES

All materials and workmanship shall be in accordance with P.D.H. Forms 408/54 and 409/49, and Section 6.24 "Supplement for Prestressed Conc. Br. Superstructures", Dated Jan. 10, 1958

Design Spec's: Division I of 1957 "Standard Specs. for Highway Bridges of AASHO except roadway slab which is designed for 16,000 lbs wheel load and fc - 1000 p.s.i.

Live Load: H-20-S-16/44 and Modified Interstate Loading (2 axles of 24 kips each @ 4 ft. c/c, 2 wheels of 12 kips @ 6 ft c/c)

Dead Loads: Includes 30 lbs. p. sq. ft. for future wearing surface on the roadway slab.

Steel Reinforcement Bars designed for fs = 18,000 psi. and detailed as per A.C.I. Code. Bars to be lapped min. 40 diameters except as noted.

2" ϕ Conduits (Galvanized Steel) shall be placed in accordance with details shown on P.D.H. Standard Dwg. SD-20

All piles are to be Cast-in-place Concrete.

Class A Concrete shall be used in curbs, parapets, pier caps, pier cols., deck slab, & backwalls above bridge seats. All other concrete shall be Class B unless otherwise noted.

The base of the footings may be ordered by the Engineer to be at any elevation or of any dimensions to provide a proper foundation.

Neoprene bearing pads, tubing and closed cellular sponge shall conform to specifications dated May 17, 1960.

Two-coat painted waterproofing shall be applied to rear faces of the walls within limits shown or as directed by the Engineer

~~Parapet, parapet railing, conduits, pull wire, scuppers and wing parapets and curbs are considered part of the superstructure.~~

Provide 2" cover on reinforcement bars unless otherwise noted.

See P.D.H. Std. Dwg. S-1614B or S-3361 for parapet railing details.

Surface of roadway divisor shall be treated as shown on P.D.H. Std. Dwg SD-13

Exposed concrete edges shall be chamfered 1"x1" unless specified or directed by the Engineer otherwise.

Use 3/8" ϕ prestressing cables with min. tendon strength = 250,000 p.s.i. ULT.

Piles shall be driven to refusal or to practical refusal as directed by the Engineer but in no case to a bearing value less than 72 tons for test piles and 60 tons for bearing piles.

SUMMARY OF QUANTITIES

ITEM	UNIT	ABUT-1	PIER-1	PIER-2	PIER-3	ABUT-2	SUPERSTR.	TOTAL
Class 1 Excavation	C.Y.	—	835	—	—	—	—	835
Class 3 Excavation	C.Y.	472	258	204	228	346	—	1508
		—	—	—	—	—		
Class A Concrete	C.Y.	—	105	98	118	—	714	1035
Class B Concrete	C.Y.	144	58	86	58	283	—	629
Reinforcement Bars	LBS	10,635	37,667	41,500	38,315	16,725	120,651	263,693
		—	—	—	—	—		
Parapet Railing	Lin. Ft.	—	—	—	—	—	510	510
Cast-in-place Conc. Piles	Lin. Ft.	—	—	719	—	—	—	719
Test Piles (Cast-in-place)	Lump Sum	—	—	2 @ 20'	—	—	—	Lump Sum
Prestressed Conc. Beams	Lin. Ft.	—	—	—	—	—	1651 (47'-48') 1050 (47'-48')	1651 (47'-48') 1050 (47'-48')
2" ϕ Galv. Steel Conduit	Lin. Ft.	—	—	—	—	—	527	527
#14 Gal. Steel Pull Wire	Lin. Ft.						548	548

Quantities shown are approximate

THESE DRAWINGS SUPERSEDE DRAWINGS S-3736 DATED JUNE 30, 1959, PREPARED BY MICHAEL BAKER JR. INC., HARRISBURG, PA.

PRESTRESS DESIGN AND REVISION PREPARED BY AMERICAN-MARIETTA CO., PITTSBURGH, PA.

JOHN R. GROVER PROFESSIONAL ENGINEER

John R. Grover
June 20, 1960

APPROVED AUG - 9 1960
K.H. Jensen
BRIDGE ENGINEER

Commonwealth of Pennsylvania

Department of Highways
BRIDGE UNIT

FOUR SIMPLE SPANS - PRESTRESS CONCRETE BRIDGE
CARRYING L.R. 187 OVER OLD WILLIAM PENN HIGHWAY
STATION 286+89.88

L.R. 187 0002289 **SECTION 12**

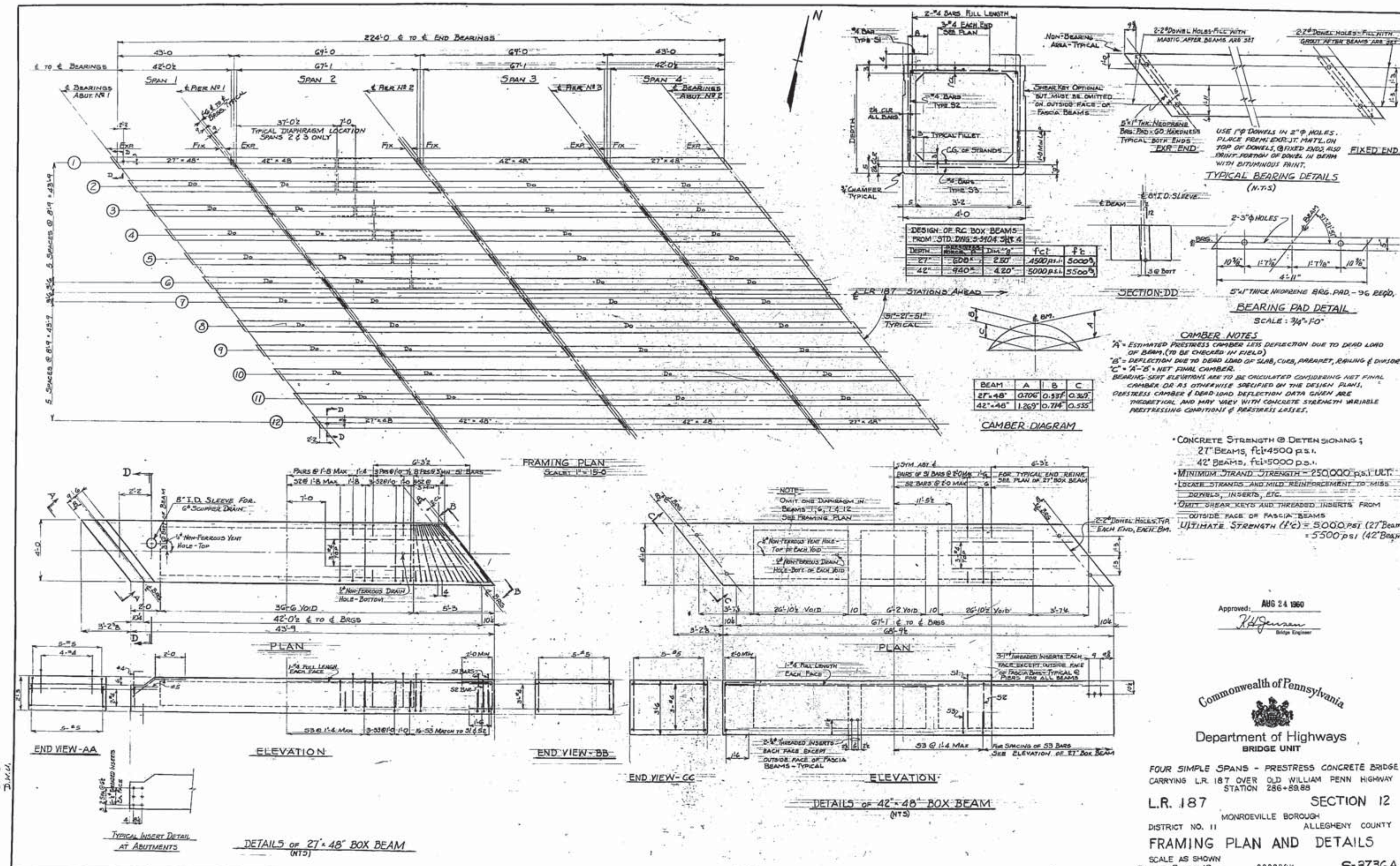
DISTRICT NO. 11 MONROEVILLE BOROUGH ALLEGHENY COUNTY

GENERAL PLAN AND ELEVATION

SCALE AS SHOWN SD 13 SD-20 S-3361 S-1614-B
SHEET 1 OF 12 **DWG. NO. S-3736A**

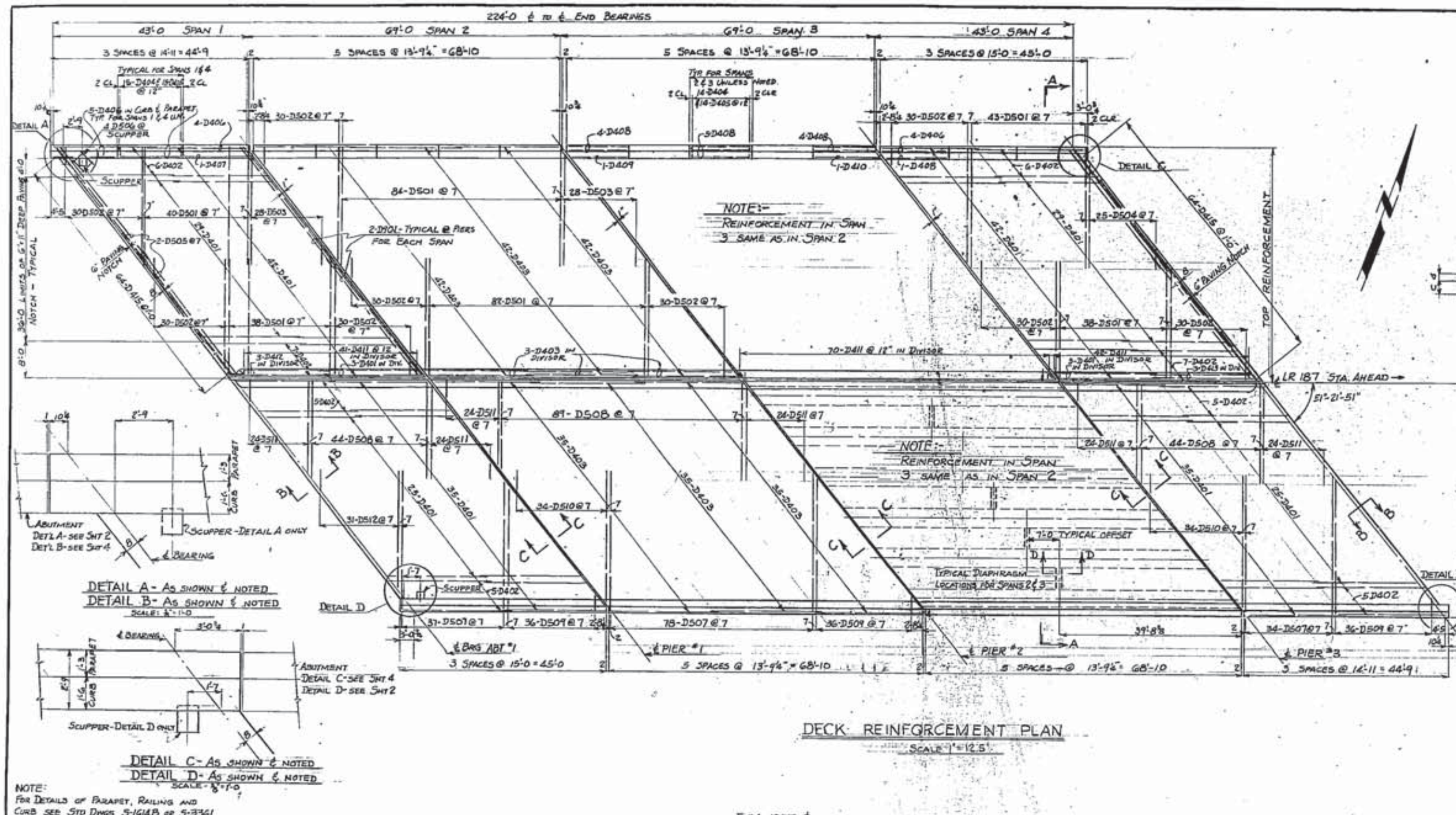
DR. | CHK. | TR. | CHD. JRG

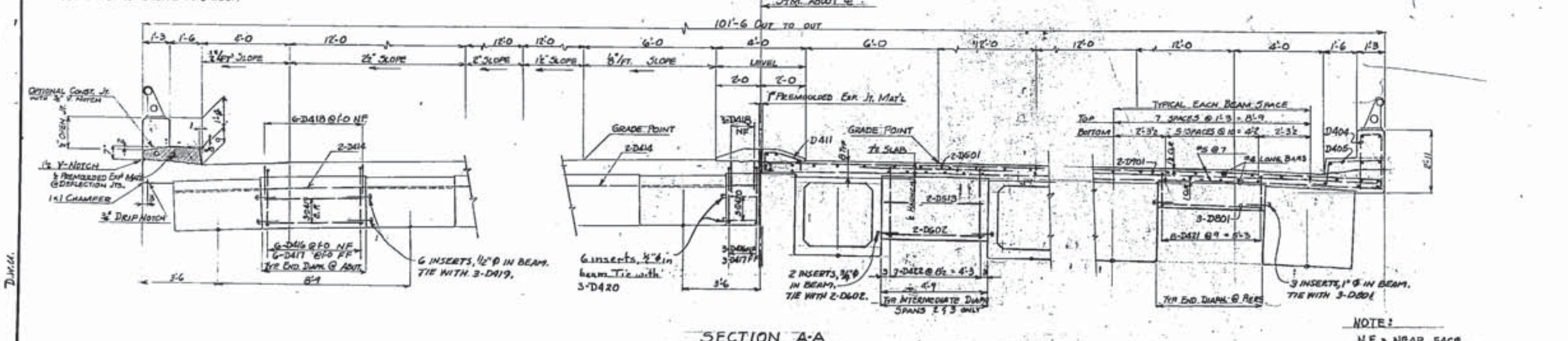
Rec'd 8-3-60



Depth	Prestress Force	Dim. "e"	f'ct	f'c
27"	600k	2.50"	4500 p.s.i.	5000 p.s.i.
42"	940k	4.20"	5000 p.s.i.	5500 p.s.i.

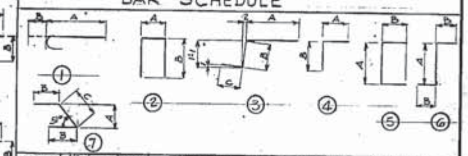
Beam	A	B	C
27" x 48"	0.706"	0.337"	0.369"
42" x 48"	1.269"	0.714"	0.555"

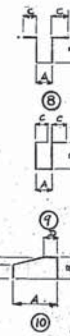




NOTE:
N.F. = NEAR FACE
F.F. = FAR FACE
E.F. = EACH FACE
WORK THIS DWG WITH SHT Nº 11

BAR SCHEDULE





MARK	REQD	SIZE	TYPE	LENGTH	A	B	C	D	REMARKS
D901	48	9	1	35'-5	34'-2	1'-3			
D801	180	8	STR	6'-0					
D601	40	6	STR	8'-9					
D602	40	6	STR	4'-6					
D501	982	5	1	26'-10	26'-3	7			
D502	18 SETS 30 EA	5	1	6'-10 to 26'-0	4'-3 to 25'-5	7			VARY EA. BAR BY 8¾
D503	6 SETS 28 EA	5	1	5'-3 to 25'-0	4'-8 to 24'-5	7			VARY EA. BAR BY 8¾
D504	2 SETS 25 EA	5	1	5'-3 to 22'-10	4'-8 to 22'-3	7			DITTO
D505	16	5	L	33'-10	33'-3	7			
D506	8	5	STR	4'-0					
D507	454	5	STR	30'-7					
D508	532	5	STR	21'-10					
D509	8 SETS 66 EA	5	STR	4'-3 to 29'-10					VARY EA. BAR BY 8¾
D510	16 SETS 34 EA	5	STR	4'-8 to 28'-9					DITTO
D511	16 SETS 24 EA	5	STR	4'-3 to 21'-0					DITTO
D512	2 SETS 31 EA	5	STR	4'-8 to 26'-7					DITTO
D513	40	5	STR	4'-6					
D401	536	4	STR	22'-1					
D402	92	4	STR	22'-9					
D403	320	4	STR	23'-10					
D404	460	4	2	6'-3	11	2'-8			
D405	460	4	3	4'-6	2'-4	1'-1	1'-1		
D406	56	4	STR	14'-8					
D407	2	4	STR	15'-8					
D408	94	4	STR	13'-5					
D409	4	4	STR	12'-5					
D410	4	4	STR	14'-5					
D411	456	4	10	2'-11	1'-9	10	6	6	
D412	6	4	STR	19'-7					
D413	6	4	STR	21'-1					
D414	16	4	STR	31'-1					
D415	256	4	4	2'-8	1'-9	11			
D416	132	4	6	3'-11	2'-7	8			
D417	132	4	5	3'-1	1'-9	8			
D418	132	4	STR	2'-0					
D419	120	4	STR	6'-0					
D420	12	4	7	4'-2	7½	1'-8	10		
D421	480	4	9	4'-8	9	1'-5½	6		
D422	140	4	8	7'-10	7	3'-1½	6		

Approved: AUG - 9 1960
K H Jensen
Bridge Engineer

Commonwealth of Pennsylvania
Department of Highways
BRIDGE UNIT

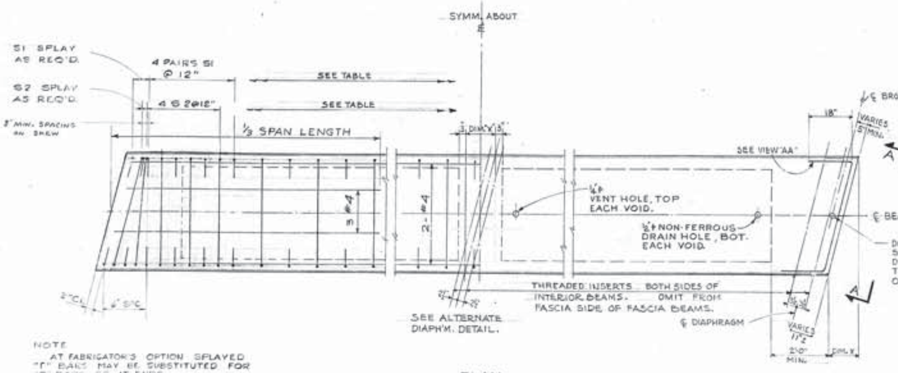
FOUR SIMPLE SPANS - PRESTRESS CONCRETE BRIDGE
CARRYING L.R. 187 OVER OLD WILLIAM PENN HIGHWAY
STATION 286+ 89.88

L.R. 187 SECTION 12
MONROEVILLE BOROUGH
DISTRICT NO. 11 ALLEGHENY COUNTY

DECK PLAN
SCALE AS SHOWN

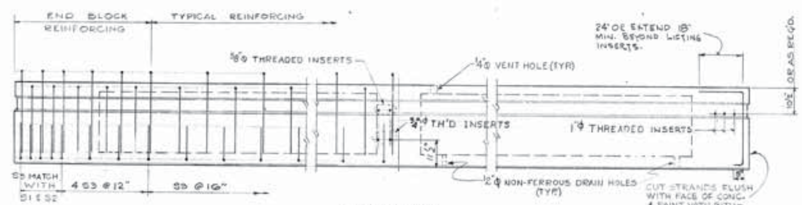
SHEET 10 OF 12 S-3736 A

0002298

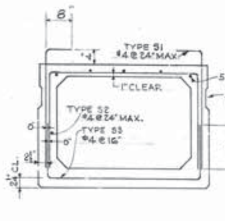


NOTE
AT FABRICATOR'S OPTION SPLAYED "T" BARS MAY BE SUBSTITUTED FOR "R" BARS S2 AT ENDS.

PLAN



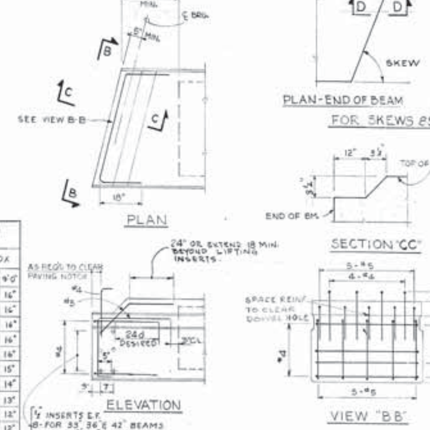
ELEVATION



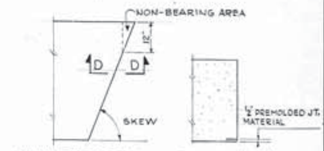
TYPICAL REINFORCMENT

VERTICAL STIRRUP REQUIREMENTS, MAXIMUM SPACING IN INCHES															
SPAN FT.	21" BOX			27" BOX			33" BOX			36" BOX			42" BOX		
	7'0"	8'0"	9'0"	7'0"	8'0"	9'0"	7'0"	8'0"	9'0"	7'0"	8'0"	9'0"	7'0"	8'0"	9'0"
30	16"	13"	11"	16"	16"	14"	16"	16"	16"	16"	16"	16"	16"	16"	16"
35	14"	11"	10"	16"	15"	12"	16"	16"	15"	16"	16"	16"	16"	16"	16"
40	12"	10"	9"	15"	13"	11"	16"	16"	13"	16"	16"	15"	16"	16"	16"
45	11"	9"	8"	14"	12"	10"	16"	15"	12"	16"	16"	14"	16"	16"	16"
50				13"	11"	9"	16"	13"	11"	16"	16"	13"	16"	16"	16"
55				12"	10"	8"	15"	12"	10"	16"	15"	13"	16"	16"	15"
60				11"	9"		14"	12"	10"	16"	14"	12"	16"	16"	14"
65				11"			13"	11"	9"	15"	13"	11"	16"	16"	13"
70							12"	10"		15"	12"	10"	16"	15"	12"
75							11"			15"	11"		16"	14"	12"
80										14"			16"	14"	
85													15"		

NOTE
VERTICAL STEEL REQUIREMETS TO BE OBTAINED BY ANY COMBINATION OF TYPE S1 & S2 STIRRUPS WITHOUT EXCEEDING MAXIMUM SPACING OF EITHER TYPE.



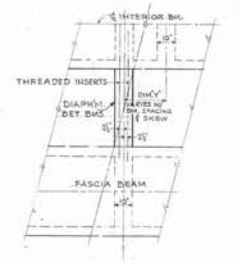
TYPICAL DETAIL AT ABUTMENT



PLAN-END OF BEAM

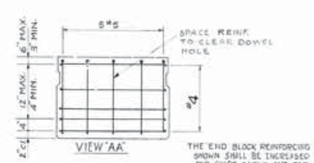
FOR SKEWS 85° OR LESS

SECTION "D D"



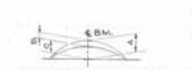
PLAN

ALTERNATE DIAPHRAGM DETAIL



VIEW "AA"

THE END BLOCK REINFORCING SHOWN SHALL BE INCREASED FOR SHARP SKEWS AND FOR HIGHLY CONCENTRATED PRESTRESSING FORCE. REARRANGEMENT OF STRANDS SHALL BE GIVEN CONSIDERATION.

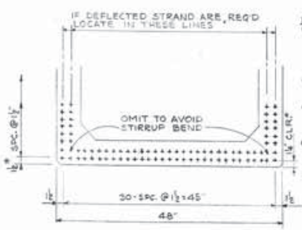


A-ESTIMATED PRESTRESS CAMBER LESS DEFLECTION DUE TO DEAD LOAD OF BEAM (TO BE CHECKED IN FIELD).
B-DEFLECTION DUE TO DEAD LOAD OF SLAB, CURB OR SIDEWALK AND PARAPET.
C-A-B-NET FINAL CAMBER.

- BEARING SEAT ELEVATION AND SLAB THICKNESS SHALL BE CALCULATED CONSIDERING A VALUE 'C'.
- THE THICKNESS OF THE HAUNCH ON BEAMS SHALL BE VARIED TO COMPENSATE FOR ANY INACCURACIES IN THE CAMBER OF BEAMS.

- PRESTRESS CAMBER AND DEAD LOAD DEFLECTION DATA SHOWN IS THEORETICAL AND MAY VARY WITH CONCRETE STRENGTH, VARIABLE PRESTRESSING CONDITIONS AND PRESTRESS LOSSES.

TYPICAL CAMBER DIAGRAM & NOTES
(TO BE SHOWN ON SHOP DWG. & DESIGN DWG.)



NOTE
- TO OBTAIN REQUIRED ECCENTRICITY OF PRESTRESSING FORCE @ ℄ SPAN, A STRAND PATTERN SHALL BE ESTABLISHED. THE TEMPLATE PATTERNS SHOWN SHALL BE USED FOR THIS PURPOSE.
- SEE TABLE OF ECCENTRICITIES ON SHEET 4
- OMIT STRANDS @ ℄ BEAM AS REQ'D BY DOWEL OR DRAINAGE ARRANGEMENT.
- STRAND PATTERN SHALL ALWAYS BE SYMMETRICAL ABOUT ℄ OF BEAM.

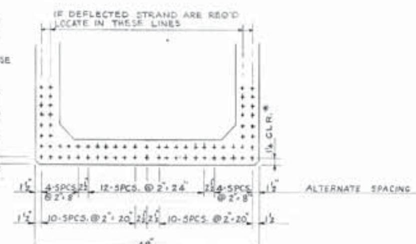
TYPICAL STRAND LAYOUT FOR 3/8"⌀ STRAND

* 1 1/4" MIN. CLEARANCE SHALL BE MAINTAINED AT ALL TIMES.

1 3/4" FROM ℄ OF STRANDS TO BOTTOM OF BEAM IS DESIRED. CHAIR WILL BE REQUIRED FOR BEAMS WITH SPANS OVER 104'-0".

1 1/2" FROM ℄ OF STRANDS TO BOTTOM OF BEAM WILL REQUIRE CHAIRS FOR SPANS OVER 50'-0" (TO ELIMINATE EXCESSIVE STRAND DRAPE).

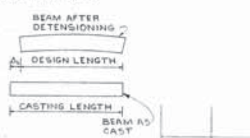
CHAIRS SHALL BE DETAILED & SHOWN ON SHOP DWGS.



TYPICAL STRAND LAYOUT FOR 7/16"⌀ STRAND

7/16"⌀ STRANDS SHALL NOT BE USED FOR SPANS LESS THAN 30'-0"

ANTICIPATED Δ_L IN.				
BOX	SPAN			
	30	50	70	90
21×48				
27×48	1/8	3/8		
33×48	0	3/8		
36×48	0	1/4	1/2	
42×48	0	1/4	1/2	



INSTRUCTION FOR CAMBER COMPUTATION

- THE CAMBER DUE TO PRESTRESS AND THE DEFLECTION DUE TO BEAM DEAD LOAD MAY BE CALCULATED USING, A 5% TO 15% LOSS IN PRESTRESS FORCE, f'ci IN CALCULATING Eci AND A CREEP FACTOR OF 1.5 TO 2.0 (IN CASE EXACT VALUES ARE NOT AVAILABLE, 5% & A FACTOR OF 2.0 IS ACCEPTABLE)
- THE DEFLECTION DUE TO SUPERIMPOSED DEAD LOADS SHALL BE CALCULATED USING, f'c IN CALCULATING Ec AND NO CREEP FACTOR.

INSTRUCTIONS

1. APPLICABLE DETAILS SHOWN ON THIS SHEET ARE TO BE INCLUDED IN FABRICATORS SHOP DWGS.
2. TOPS OF ALL BEAMS SHALL BE STRUCK OFF AND SCORED AS DIRECTED BY THE ENGINEER (SHALL BE SPECIFIED ON SHOP DRWGS).
3. ALL REINFORCING SHOWN ON THIS SHEET IS FOR INTERMEDIATE GRADE STEEL @ f_s = 20,000 P.S.I., IF STRUCTURAL GRADE STEEL IS USED, INCREASE STEEL REQUIREMENTS OF STIRRUPS BY 10%. SPECIFY ON SHOP DRAWING STEEL TO BE USED.
4. THIS SHEET TO BE WORKED WITH SHEETS 3 & 4.
5. PRESTRESS CAMBER & DEAD LOAD DEFLECTIONS SHALL BE SHOWN ON THE SHOP DWGS. & DESIGN PLANS.
6. SHOW TYPICAL CROSS SECTION & FRAMING PLAN OF BRIDGE ON FABRICATORS SHOP DWG.
7. SHOW BEARING PAD DETAILS ON FABRICATORS SHOP DWG.
8. DESIGN LENGTHS SHALL BE SHOWN ON SHOP DWGS., CASTING LENGTH MAY BE GIVEN IN ( ).
9. ALL INSERTS MAY BE CAST IN BEAM OR SELF-DRILLING ANCHORS. TYPE OF ANCHOR SHALL BE SPECIFIED ON SHOP DWGS.
10. FOR BEAM FABRICATION TOLERANCE, SEE SHEET 1.

NO.	DESCRIPTION	BY	CHKD.	DATE

REVISIONS

APPROVED SEP 19 1960

BRIDGE ENGINEER

Commonwealth of Pennsylvania

Department of Highways

BRIDGE UNIT

PRESTRESSED CONCRETE BRIDGE STANDARDS

4 FT. SPREAD BOX BEAMS

COMPOSITE

BEAM DETAILS

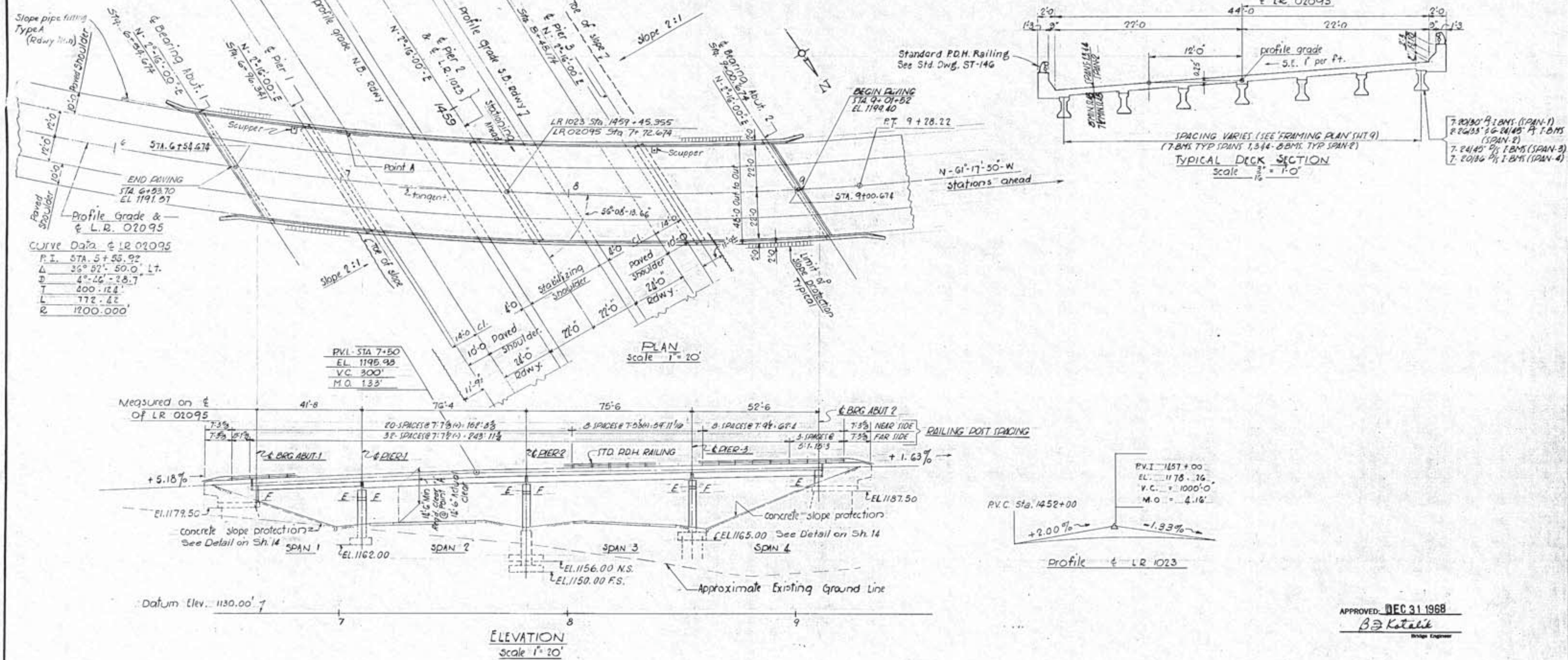
SCALE: NO SCALE
SHEET 5 OF 6

S-3904

PREPARED BY SCHUPACK AND ZOLLMAN, NEWTOWN SQUARE, IN COOPERATION WITH THE PRESTRESSED CONCRETE ASSOCIATION OF PENNSYLVANIA FOR THE PENNSYLVANIA DEPARTMENT OF HIGHWAYS.

C.3 Bridge K

Structural drawings for bridge K (PADoH 1960a and 1960b).



APPROVED: DEC 31 1968

B. E. Katalik

Bridge Engineer

DEPARTMENT OF HIGHWAYS

BRIDGE DIVISION

ALLEGHENY COUNTY

BEAVER VALLEY EXPRESSWAY

L.R. 1023, SEC. 1A, STA. 1459+45.355

UNDER L.R. 02095

4-SPAN PS CONCRETE I-BEAM BRIDGE

SUPERSTRUCTURE & SUBSTRUCTURE

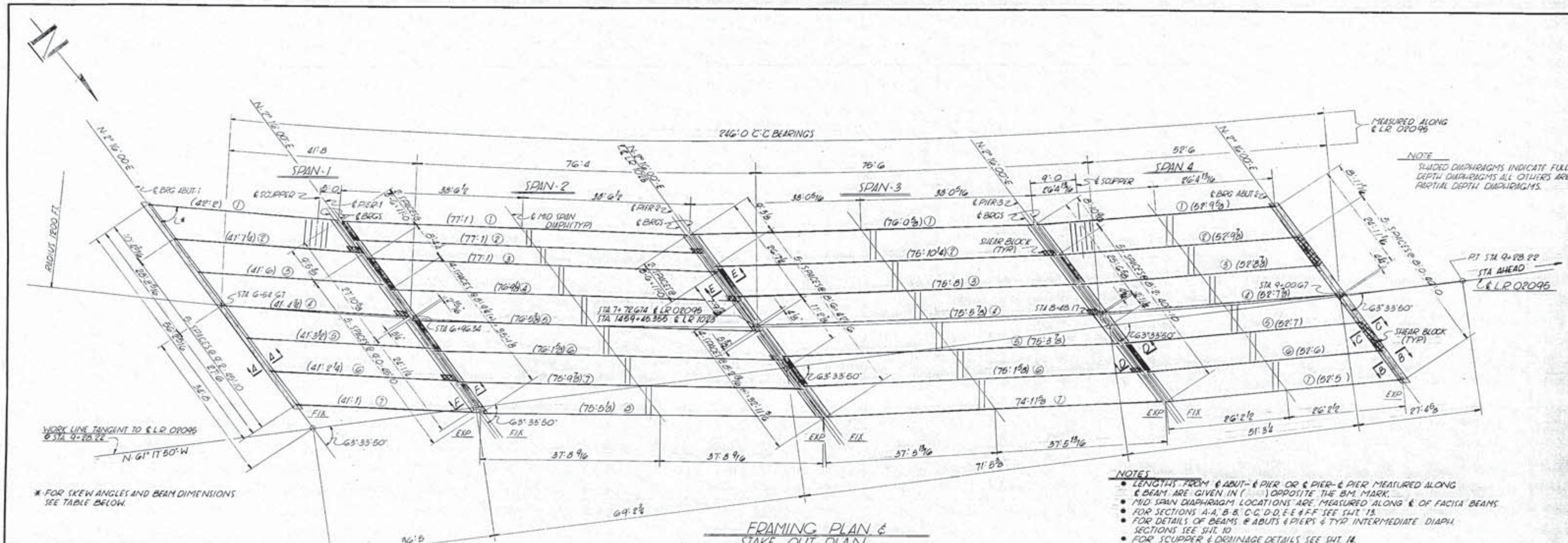
GENERAL PLAN

SCALE: AS NOTED

0004057

SHEET 2 OF 16

S-7894A



NOTE
SHADED DIAPHRAGMS INDICATE FULL DEPTH DIAPHRAGMS ALL OTHERS ARE PARTIAL DEPTH DIAPHRAGMS.

* FOR SKEW ANGLES AND BEAM DIMENSIONS SEE TABLE BELOW.

FRAMING PLAN & STAKE OUT PLAN

SCALE 1"-10'-0

NOTES
- LENGTHS FROM ℄ ABUT-℄ PIER OR ℄ PIER-℄ PIER MEASURED ALONG ℄ BEAM ARE GIVEN IN ( ) OPPOSITE THE BM. MARK.
- MID-SPAN DIAPHRAGM LOCATIONS ARE MEASURED ALONG ℄ OF FACISA BEAMS.
- FOR SECTIONS A-A, B-B, C-C, D-D, E-E & F-F SEE SHT. 13.
- FOR DETAILS OF BEAMS @ ABUTS & PIERS & TYP INTERMEDIATE DIAPH. SECTIONS SEE SHT. 10
- FOR SCUPPER & DRAINAGE DETAILS SEE SHT. 14.

BEAM DIMENSIONS

SPAN	BM. No	BM. SIZE	C-C BEARINGS	TOTAL BM LENGTH	SKEW ANGLE
1	1	20/30	41'-1 1/8	42'-6 1/2	50° 38' 27"
	2		40'-6 1/2	41'-11 3/4	51° 36' 42"
	3		40'-5 1/4	41'-10 1/2	51° 47' 35"
	4		40'-4	41'-9 1/4	51° 58' 33"
	5		40'-2 7/8	41'-8	52° 09' 29"
	6		40'-1 5/8	41'-6 3/4	52° 20' 32"
	7	20/30	40'-0 3/8	41'-5 1/2	52° 31' 37"
2	1	26/33	75'-0 1/8	76'-9	53° 32' 15"
	2	26/33	75'-0 1/8	76'-7	53° 32' 15"
	3	24/45	75'-0 1/8	76'-7	53° 32' 15"
	4		74'-8 3/8	76'-3 1/8	53° 51' 35"
	5		74'-4 3/4	75'-11 1/8	54° 11' 10"
	6		74'-1 1/8	75'-7 5/8	54° 30' 52"
	7		73'-9 1/2	75'-3 1/8	54° 50' 44"
	8	24/45	73'-4 7/8	75'-1 1/8	55° 10' 46"
3	1	24/45	74'-0 1/8	75'-8 5/8	57° 16' 00"
	2		73'-10 5/8	75'-4 1/4	57° 30' 33"
	3		73'-8 3/8	75'-2	57° 43' 19"
	4		73'-6 1/8	74'-11 7/8	57° 56' 08"
	5		73'-4 3/8	74'-9 7/8	58° 09' 01"
	6		73'-2 1/8	74'-7 5/8	58° 21' 55"
	7	24/45	73'-0 1/4	74'-7 1/8	58° 34' 58"
4	1	20/36	51'-10 1/8	53'-2 1/8	60° 23' 52"
	2		51'-10 1/8	53'-2 1/8	60° 21' 30"
	3		51'-9 1/8	53'-1 1/8	60° 30' 57"
	4		51'-8 3/8	53'-0 3/8	60° 40' 26"
	5		51'-7 5/8	52'-11 1/2	60° 49' 58"
	6		51'-6 5/8	52'-10 1/2	60° 59' 31"
	7	20/36	51'-5 5/8	52'-9 1/2	61° 09' 05"

V.C. CORRECTION
.025' SPAN-1
.086' SPAN-2
.084' SPAN-3
.041' SPAN-4

S.E. CORRECTION
.063' SPAN-1
.119' SPAN-2 (26/33")
.112' SPAN-2 (24/45")
.111' SPAN-3
.072' SPAN-4

A-ESTIMATED PRESTRESS CAMBER LESS DEFLECTION DUE TO DEAD LOAD OF BEAM (TO BE CHECKED IN FIELD)

B-DEFLECTION DUE TO DEAD LOAD OF SLAB, CURB & PARAPET.

C-A-B NET FINAL CAMBER.

- NET FINAL CAMBER, VERT. CURVE & S.E. CORRECTION HAS BEEN INCLUDED IN THE CALCULATIONS FOR BM. SEAT ELEVATIONS AND HAUNCH THICKNESS @ ℄ BRG
- THE THICKNESS OF THE HAUNCH ON THE BEAMS SHALL VARY TO COMPENSATE FOR ANY IRREGULARITIES IN THE CAMBER OF THE BEAMS.
- PRESTRESS CAMBER AND DEAD LOAD DEFLECTION DATA SHOWN IS THEORETICAL AND MAY VARY WITH CONCRETE STRENGTH, VARIABLE PRESTRESSING CONDITIONS, AND PRESTRESS LOSSES.

CAMBER DIAGRAM

STRAND DEF'L DETAIL

(N.T.S.)

PRESTRESS LOAD AND CAMBER TABLE

SPAN	BM SIZE	A	B	C	f'c	f'ci	Fi	g @ ℄	g @ END	X	BEARING PADS FIXED END	BEARING PADS EXP END
1	20/30	0.848"	0.295"	0.553"	5500	5000	521K	6.68"	6.68	~	9" x 5/8" x 18"	9" x 5/8" x 18"
2	26/33	2.710"	0.983"	1.727"	6000	5150	1172K	7.02"	11.10"	11'-2 1/2	9" x 1/2" x 22"	9" x 1 1/4" x 22"
	24/45	2.602"	0.729"	1.873"	5500	5000	934K	5.24"	11.40	12'-6 ±	10" x 1/2" x 22"	10" x 1 1/4" x 22"
3	24/45	2.115"	0.726	1.389"	5500	5000	955K	5.58	11.38	12'-6 ±	11" x 3/4" x 22"	11" x 1 1/8" x 22"
4	20/36	0.785"	0.342	0.443"	5500	5000	716K	8.41	8.41	~	10" x 1/2" x 18"	10" x 3/4" x 18"

f'c = 28 DAY CLY. STRENGTH & DESIGN STRENGTH OF CONCRETE.
f'ci: CONCRETE STRENGTH @ RELEASE OF PRESTRESS.
Fi - TOTAL INITAIL PRESTRESSING FORCE PER BEAM.
g - DISTANCE FROM BEAM SOFFIT TO C.G. OF STRANDS.
X = DISTANCE FROM ℄ SPAN TO STRAND DEF'L PT. (SEE DETAIL).
- ALL BEARING PADS SHALL BE 60 DUROMETER NEOPRENE.

PRESTRESS NOTES

- ALL MATERIAL AND WORKMANSHIP SHALL BE IN ACCORDANCE WITH SECT. 6.14 SUPPLEMENT FOR "PRESTRESS CONC BRIDGE SUPERSTRUCTURE" DATED JUNE 22, 1966 OF P.D.H. FORM 408-60.
- PRESTRESS BEAM REINF. AND DETAILS SHALL BE SHOWN ON FABRICATORS SHOP DWGS. AND WILL CONFORM TO THE LATEST P.D.H. REQUIRMENTS.

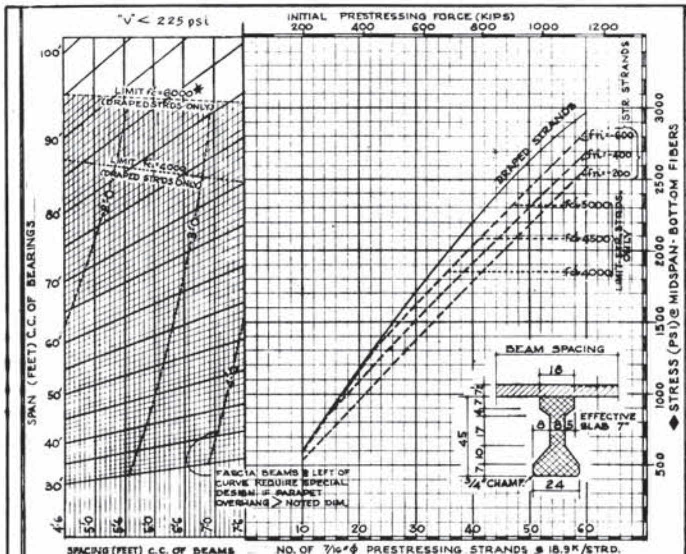
REFER TO P.D.H. PRESTRESS CONC. BRIDGE STDS. ST-208 (APPR. AUG 17, 1964.)

- REFER TO SHEETS 5, 6 & 8 FOR: DESIGN NOTES AND TYP SECTIONS FOR 20/30", 26/33", 24/45" & 20/36" I-BMS.
- REFER TO SHEET 9 FOR: TYPICAL CAMBER DIAGRAM AND NOTES EXCEPT AS SUPPLEMENTED ON THIS DWG. BEAM FABRICATION DETAILS, AND DETAILS OF MILD STEEL REINF. (MAX STIRRUP SPA. SHALL BE 12" ALL SPANS).

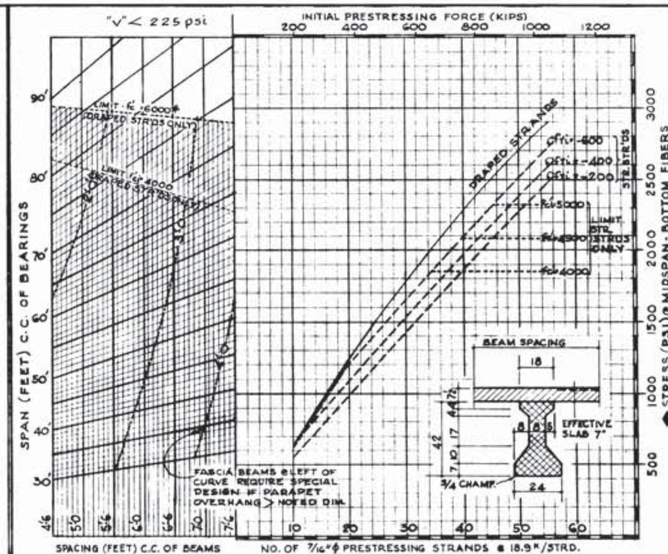
APPROVED: DEC 31 1968
B. J. Ketzel
Bridge Engineer

Commonwealth of Pennsylvania
DEPARTMENT OF HIGHWAYS
BRIDGE DIVISION
ALLEGHENY COUNTY
BEAVER VALLEY EXPRESSWAY
L.R. 1023 SEC. 1A STA. 1459+45.355
UNDER L.R. 02095
4-SPAN P/S CONCRETE I-BEAM BRIDGE
SUPERSTRUCTURE
FRAMING PLAN

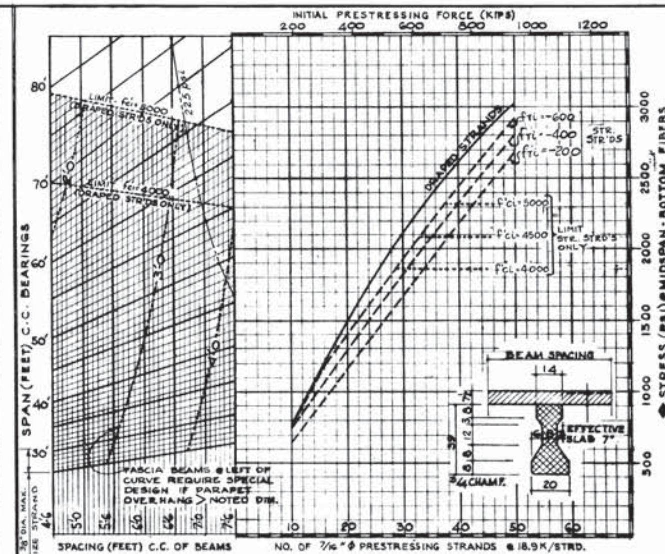
SCALE: 1"-10'-0
SHEET 9 OF 16
0004064
S-7894-A



BEAM 24/45



BEAM 24/42



BEAM 20/39

♦ SEE NOTE - SHEET 4

*TOP FIBER AT MIDSPAN UNDER FULL LOAD CONTROLS TO ESTABLISH LIMIT OF 28 DAY CYL. STRENGTH (f'c) USE f'ci = 5000 PSI

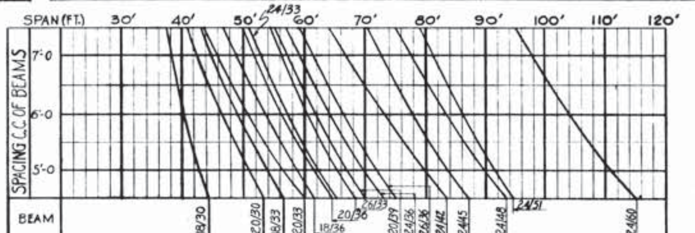
SECTION PROPERTIES

BEAM SIZE	BASIC SECTION						COMPOSITE SECTION													
							4'-6 C.C.		5'-0 C.C.		5'-6 C.C.		6'-0 C.C.		6'-6 C.C.		7'-0 C.C.		7'-6 C.C.	
	CONC.	A	y_b	I	Z_t	Z_b	y_b	I	y_b	I	y_b	I	y_b	I	y_b	I	y_b	I	y_b	I
UNIT	yd³/ft	IN²	IN	IN⁴	IN³	IN³	IN	IN⁴	IN	IN⁴	IN	IN⁴	IN	IN⁴	IN	IN⁴	IN	IN⁴	IN	IN⁴
24/45	0.165	642	20.18	140,129	5,646	6,944	30.65	332,424	31.38	345,475	32.03	357,597	32.64	368,178	33.20	378,607	33.72	388,636	34.21	397,353
24/42	0.151	588	18.04	107,986	4,507	5,986	28.80	282,931	29.48	294,443	30.12	305,157	30.72	314,147	31.26	323,830	31.77	332,077	32.24	340,240
20/39	0.132	513	17.41	77,514	3,590	4,452	28.10	216,059	28.70	224,603	29.30	232,327	29.84	239,951	30.35	245,879	30.81	252,343	31.24	257,999

DIMENSION "e" (INCHES)

FOR 7/16"φ STRANDS

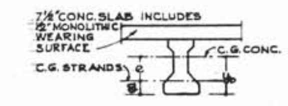
Nº OF 7/16" STRANDS	BEAM 24/45				BEAM 24/42				BEAM 20/39			
	DRAPED STR'DS	STRAIGHT STR'DS: fti =			DRAPED STR'DS	STRAIGHT STR'DS: fti =			DRAPED STR'DS	STRAIGHT STR'DS: fti =		
		-600	-400	-200		-600	-400	-200		-600	-400	-200
10	2.00	2.00	2.00	5.40	2.00	2.00	2.00	5.62	2.40	2.40	2.81	6.62
15	2.67	2.67	3.46	7.42	2.67	2.67	4.04	7.14	2.93	2.93	5.31	7.89
20	3.00	3.00	5.38	8.65	3.00	3.23	5.64	7.99	3.50	4.67	6.61	8.51
25	3.52	4.16	6.68	9.00	3.52	4.61	6.54	8.45	4.00	5.85	7.31	8.87
30	3.93	5.40	7.38	9.36	3.93	5.62	7.14	8.75	4.60	6.62	7.89	9.14
35	4.40	6.28	7.98	9.71	4.40	6.31	7.64	8.99	5.20	7.11	8.23	9.29
40	4.85	6.89	8.38	9.88	4.85	6.82	8.04	9.16	5.90	7.53	8.51	9.42
45	5.38	7.42	8.78	10.07	5.38	7.22	8.27	9.30	6.67	7.88	8.72	9.54
50	5.92	7.79	8.98	10.22	5.92	7.53	8.45	9.44	7.60	8.10	8.86	9.63
55	6.55	8.15	9.18	10.33	6.55	—	—	—	—	—	—	—
60	7.35	8.39	9.38	10.42	—	—	—	—	—	—	—	—



SPANS (C.C BRGS) FOR VARIOUS BEAMS* - DRAPED STRANDS

FOR DESIGN NOTES SEE SHEET 4

*USE CURVES FOR PRELIMINARY SELECTION OF BEAM SIZE ONLY.



INSTRUCTIONS:
1· FOR CONSTRUCTION PURPOSES SOME STRANDS MAY BE LOCATED IN UPPER FLANGE OR WEB OF BEAM. SUCH STR'DS SHALL BE INCLUDED IN COMPUTING THE CENTER OF GRAVITY OF STRANDS IF PRESTRESSED

2· VALUE "e" FOR Nº OF STRANDS OTHER THAN THAT GIVEN IN TABLE TO BE INTERPOLATED AND NEAREST PRACTICAL VALUE TO BE USED.

APPROVED AUG 17 1964
K.H. Jensen
BRIDGE ENGINEER

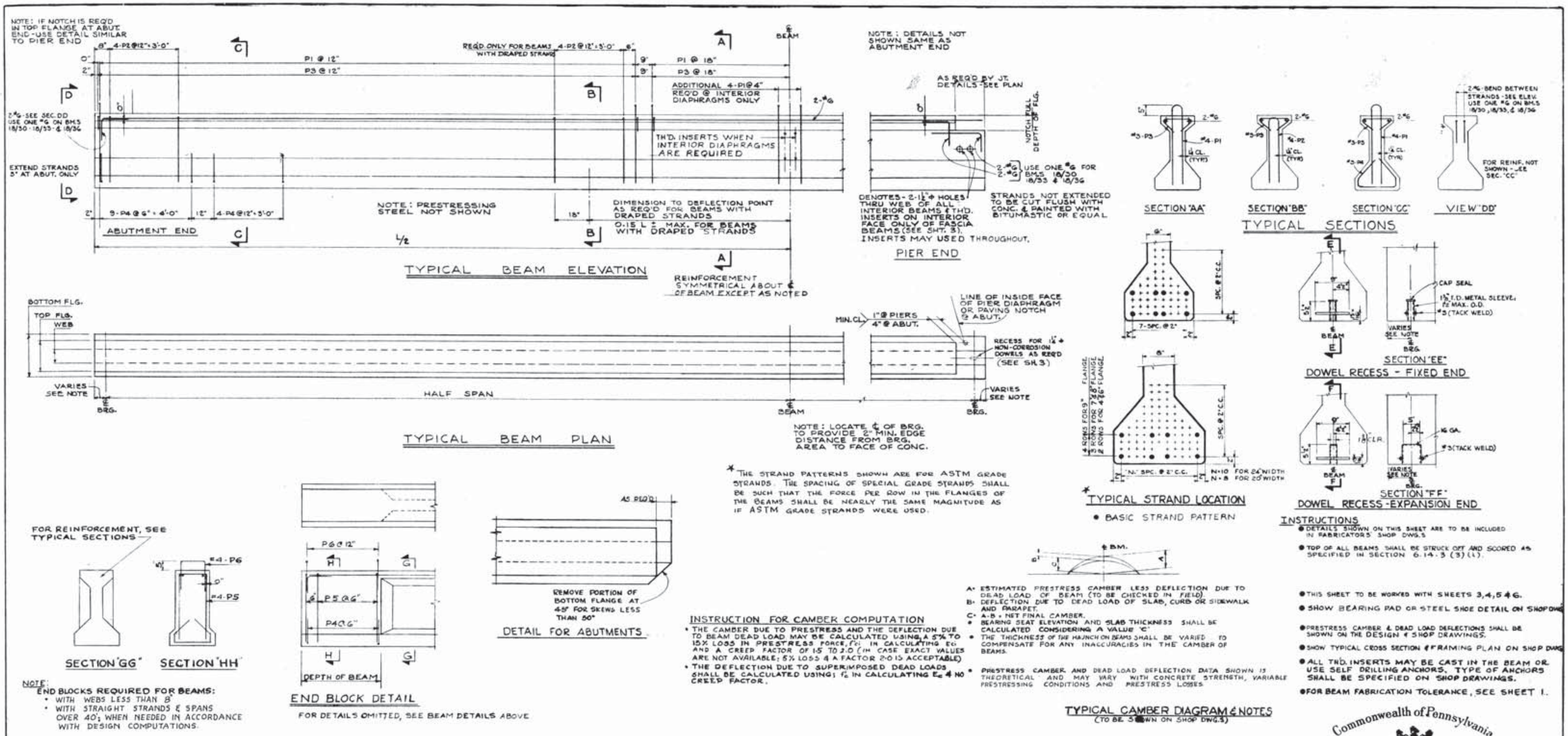
Commonwealth of Pennsylvania
Department of Highways
BRIDGE UNIT
PRESTRESSED CONCRETE BRIDGE - STANDARDS

"I" BEAMS
COMPOSITE
DESIGN CURVES
24/45 24/42 20/39

SHEET 5 OF 11

ST-208

PREPARED BY SCHUPACK & ZOLLMAN, NEWTOWN SQUARE, PA. FOR PENNSYLVANIA DEPARTMENT OF HIGHWAYS - CENTRAL OFFICE BRIDGE UNIT



ANTICIPATED Δ_L INCHES				
BEAM	SPAN			
	40	60	80	100
ALL BM.				

NOTE:
END BLOCKS REQUIRED FOR BEAMS:
- WITH WEBS LESS THAN 8"
- WITH STRAIGHT STRANDS & SPANS OVER 40'; WHEN NEEDED IN ACCORDANCE WITH DESIGN COMPUTATIONS.

* THE STRAND PATTERNS SHOWN ARE FOR ASTM GRADE STRANDS. THE SPACING OF SPECIAL GRADE STRANDS SHALL BE SUCH THAT THE FORCE PER ROW IN THE FLANGES OF THE BEAMS SHALL BE NEARLY THE SAME MAGNITUDE AS IF ASTM GRADE STRANDS WERE USED.

INSTRUCTION FOR CAMBER COMPUTATION
- THE CAMBER DUE TO PRESTRESS AND THE DEFLECTION DUE TO BEAM DEAD LOAD MAY BE CALCULATED USING A 5% TO 15% LOSS IN PRESTRESS FORCE, f'_{ci} IN CALCULATING E_{ci} AND A CREEP FACTOR OF 1.5 TO 2.0 (IN CASE EXACT VALUES ARE NOT AVAILABLE; 5% LOSS & A FACTOR 2.0 IS ACCEPTABLE)
- THE DEFLECTION DUE TO SUPERIMPOSED DEAD LOADS SHALL BE CALCULATED USING; f'_c IN CALCULATING E_c & NO CREEP FACTOR.

A- ESTIMATED PRESTRESS CAMBER LESS DEFLECTION DUE TO DEAD LOAD OF BEAM (TO BE CHECKED IN FIELD).
B- DEFLECTION DUE TO DEAD LOAD OF SLAB, CURB OR SIDEWALK AND PARAPET.
C- A-B = NET FINAL CAMBER.
- BEARING SEAT ELEVATION AND SLAB THICKNESS SHALL BE CALCULATED CONSIDERING A VALUE "C"
- THE THICKNESS OF THE HAUNCH ON BEAMS SHALL BE VARIED TO COMPENSATE FOR ANY INACCURACIES IN THE CAMBER OF BEAMS.
- PRESTRESS CAMBER AND DEAD LOAD DEFLECTION DATA SHOWN IS THEORETICAL AND MAY VARY WITH CONCRETE STRENGTH, VARIABLE PRESTRESSING CONDITIONS AND PRESTRESS LOSSES.

INSTRUCTIONS
- DETAILS SHOWN ON THIS SHEET ARE TO BE INCLUDED IN FABRICATORS' SHOP DWG.S
- TOP OF ALL BEAMS SHALL BE STRUCK OFF AND SCORED AS SPECIFIED IN SECTION 6.14.3 (3) (L).
- THIS SHEET TO BE WORKED WITH SHEETS 3,4,5 & 6.
- SHOW BEARING PAD OR STEEL SHOE DETAIL ON SHOP DWG
- PRESTRESS CAMBER & DEAD LOAD DEFLECTIONS SHALL BE SHOWN ON THE DESIGN & SHOP DRAWINGS.
- SHOW TYPICAL CROSS SECTION & FRAMING PLAN ON SHOP DWG
- ALL THD. INSERTS MAY BE CAST IN THE BEAM OR USE SELF DRILLING ANCHORS. TYPE OF ANCHORS SHALL BE SPECIFIED ON SHOP DRAWINGS.
- FOR BEAM FABRICATION TOLERANCE, SEE SHEET 1.

Commonwealth of Pennsylvania
Department of Highways
BRIDGE UNIT
PRESTRESSED CONCRETE BRIDGE STANDARDS
"I" BEAMS
COMPOSITE
BEAM DETAILS

APPROVED AUG 17 1964
BRIDGE ENGINEER
SCALE: NO SCALE
SHEET 9 OF 11
ST- 208

PREPARED BY SCHUPACK AND ZOLLMAN, NEWTOWN SQUARE, IN COOPERATION WITH THE PRESTRESSED CONCRETE ASSOCIATION OF PENNSYLVANIA FOR THE PENNSYLVANIA DEPARTMENT OF HIGHWAYS.