23. Bridge Rehabilitation

23.1. Introduction

Properly timed bridge maintenance and rehabilitation can maximize the service life of a bridge and delay the need for its replacement. This work will minimize the probability that bridges will deteriorate to an unsafe or unserviceable condition and protects the large capital investment in Alaska's inventory of bridges.

23.1.1. Scope of Work Definitions

Section 10.2.3 presents scope of work definitions to distinguish between the various levels of bridge work. Specifically, Section 10.2.3 presents definitions of the following:

- major bridge rehabilitation,
- minor bridge rehabilitation,
- bridge deck rehabilitation/replacement,
- seismic retrofit, and
- bridge widening.

Section 28.3 of this *Manual* discusses FHWA funding eligibility for bridge rehabilitation projects.

23.1.2. Rehabilitation Strategy

The development of a bridge rehabilitation project involves the following basic steps:

1. Collect the available data on the existing bridge (e.g., as-built plans, bridge inspection reports, load ratings, traffic volumes).
2. Perform a field investigation of the existing bridge. This may or may not be necessary.
3. Identify the necessary condition surveys and tests (e.g., coring, chain drag, chloride analysis, identifying fracture-critical members).
4. Evaluate the data from the condition surveys and tests.
5. Identify feasible rehabilitation strategies, estimate their costs, and compare these costs to the anticipated benefits.
6. Select and document the appropriate bridge rehabilitation strategy to upgrade the bridge to meet the necessary structural and functional objectives.
7. After the preliminary plans are completed, it is frequently a good idea to perform a field review to verify the design.

23.1.3. 3R Projects

Many bridge rehabilitation projects are identified as part of a highway 3R project.

For 3R projects, structural retrofits are required for any existing structural member with a capacity less than HS15 or HS20 for interstate bridges.

In addition to the 3R requirements in this *Manual*, follow the requirements in Section 1160.3.5 of the *Alaska Highway Preconstruction Manual*.

23.2. Documentation

23.2.1. Field Inspection

After reviewing relevant background material (e.g., as-built plans, shop drawings, bridge inspection reports, SI&A data, traffic data), the bridge engineer may determine that a field inspection is warranted. One objective is to identify the various condition tests and surveys that may be needed. Review the as-built plans before the field inspection.

During the field inspection note any areas of special concern (e.g., delamination, fatigue-critical details, bridge rail). Take photographs of approaches, elevation view, all four quadrants of the bridge, the feature being crossed, and any deficient features.

Ensure that all information is gathered as necessary to complete the Bridge 3R Memo.

In addition, verify that the bridge details match those shown in the as-built plans and shop drawings. Also, check for evidence of repair work or revisions not indicated in the plans and shop drawings.

Arrange for testing for the presence of lead-based paints and mill scale on steel structures.

23.2.2. Bridge 3R Memo

Figure 23-1 presents a sample bridge 3R memo, which should:

- document the findings from the field inspection, if conducted, including photographs;
- identify deficient items and provide recommendations for upgrade or repair;
- document the recommendations for seismic retrofit;
- make recommendations on the proposed bridge rehabilitation improvements;
- note scour susceptibility and provide a recommendation for upgrade or repair, if appropriate; and
- provide estimated design and construction project cost estimates.

Submit the bridge 3R memo to the project manager in the appropriate regional office.

23.2.3. Bridge Rehabilitation Literature

The design of new bridges is based primarily on the AASHTO *LRFD Bridge Design Specifications*. No single national publication exists that presents accepted practices, policies, and criteria for the rehabilitation of existing bridges as the *LRFD Specifications* provide for original design. However, the highway research community has devoted significant resources to identify practical, cost-effective methods to rehabilitate existing highway bridges.

Publications are readily available that may be of special interest when rehabilitating an existing bridge. Figure 23-2 provides a list of some of the more prominent publications that may be useful on a project-by-project basis.

MEMORANDUM

State of Alaska

Department of Transportation & Public Facilities
Statewide Design & Engineering Services Division/Bridge Section

TO:	Tiff Vincent Project Engineer Northern Region	**DATE:**	May 12, 2006
		FILE:	232, 233, 234, 235, 237, 238, 239
		TELEPHONE:	465-2975
		FAX:	465-6947
FROM:	Richard A. Pratt, P.E. Chief Bridge Engineer	**TEXT TELEPHONE:**	465-3652
CONTACT:	Drew Sielbach Bridge Management Engineer	**SUBJECT:**	AKSAS 61425 3R Evaluation, Chena Hot Springs Road MP 22-54

The estimated bridge 3R construction cost is $1,369,000. This includes 10% Mobilization and Demobilization costs, 15% Contingency, and 15% Construction Engineering. Recommended work includes:

North Fork Chena River, BN 232:

- Widen the pier cap.
- Install new bridge railing.
- Place backfill under both abutment caps.
- Install Name Place and Object Marker signs at both bridge ends.

North Fork Chena River, BN 233:

- Widen the pier cap.
- Install new bridge railing.
- Place backfill under one abutment cap.
- Install Name Place and Object Marker signs at both bridge ends.

North Fork Chena River, BN 234:

- Widen the pier caps.
- Install new bridge railing.
- Install Name Place and Object Marker signs at both bridge ends.

North Fork Chena River, BN 235:

- Widen the pier cap.
- Install new bridge railing.
- Install Name Place and Object Marker signs at both bridge ends.

"Providing for the movement of people and goods and the delivery of state services."

Figure 23-1 — Sample Bridge 3R Memo
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North Fork Chena River, BN 237:

- Widen the pier caps.
- Install new bridge railing.
- Install Name Place and Object Marker signs at both bridge ends.

Angel Creek, BN 238:

- Install new bridge railing.
- Install Name Place and Object Marker signs at both bridge ends.

West Fork Chena River, BN 239:

- Widen the pier cap.
- Install new bridge railing.
- Place backfill under one abutment cap.
- Install Name Place and Object Marker signs at both bridge ends.

The 3R bridge analysis and cost estimates are attached.

Costs associated with seismic retrofit ($640,000.00) are eligible for funding using Federal Bridge Funds. Costs associated with all other work activities are not eligible for funding using Federal Bridge Funds.

The estimated Bridge PS&E development cost is $150,000. We believe it will take 6 to 8 months to develop the PS&E.

Figure 23-1 — Sample Bridge 3R Memo
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3R Bridge Analysis

BRIDGE NAME	BRIDGE NUMBER	MILEPOINT	SUFFICIENCY RATING	LENGTH (Feet)	WIDTH (Feet)
North Fork Chena River	232	37.8	91.2	162.1	27.9

3R Criteria Evaluated

Width: The bridge has a design ADT of 247 and the usable bridge deck width is greater than the required 3R width. Based on this information, the existing bridge deck width satisfies the 3R criteria. Therefore, widening is not required.

Structural Capacity: The Preconstruction Manual states, "If any existing structural member has a design capacity less than HS 15 (HS 20 for interstate bridges), replace that member." The Chena Hot Springs Road is not classified as an "interstate" highway and therefore, must satisfy HS 15 criteria. The bridge inventory ratings are greater than HS 20. Therefore, no strengthening is required.

Bridge Rail and Transitions: Inspection reports indicate the presence of missing, loose, and cut bridge rail anchor bolt nuts. A file review found that the 1990 rail retrofit design decreased the available nut installation width, which inadvertently required field modification of the anchorage system. The amount of field modification and affect on bridge rail performance cannot be calculated. Therefore, installation of new bridge rail and transitions are required.

Note: If the Alaska Multi-State Bridge Rail is used, then the usable bridge deck width will decrease from 28'-0" to 26'-7". This reduced usable bridge deck width exceeds the minimum required 3R width.

Earthquake Capacity: This bridge does not satisfy the 3R seismic bearing width criteria. Therefore, seismic retrofit, pier cap widening, is required.

Other: A purpose of 3R projects is to correct deficiencies identified by the Bridge Inspection Program and Maintenance and Operations. Recent inspections identify the following work items:

- Place backfill under both abutment caps to retain approach fill.
- Install Name Place and Object Marker signs at both bridge ends.

Cost Summary: The estimated rehabilitation construction costs follow:

Item No.	Item Description	Unit	Qty	Total Cost
205(4)	Porous Backfill	CY	20	$6,000
401(1)	Class A Concrete	CY	6	$14,000
501(10)	Coring Concrete	LF	75	$18,000
502(1)	Post Tensioning (CIP Concrete)	Each	28	$9,800
503(1)	Reinforcing Steel	LB	1,800	$6,000
504(1)	Structural Steel	LB	1,000	$7,500
507(1)	Steel Bridge Rail	LF	324	$64,800
606(12)	Guardrail/Bridge Rail Connection	EA	4	$8,000
615(1)	Standard Sign	SF	42	$3,500
640(1)	Mobilization and Demobilization	LS	(10%)	$13,800
	Contingencies		(15%)	$22,800
	Construction Engineering		(15%)	$22,800
TOTAL:				**$197,000**

Figure 23-1 — Sample Bridge 3R Memo
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Tiff Vincent 4 May 11, 2006

BRIDGE NAME	BRIDGE NUMBER	MILEPOINT	SUFFICIENCY RATING	LENGTH (Feet)	WIDTH (Feet)
North Fork Chena River	233	39.5	91.2	162.1	27.9

3R Criteria Evaluated

Width: The bridge has a design ADT of 247 and the usable bridge deck width is greater than the required 3R width. Based on this information, the existing bridge deck width satisfies the 3R criteria. Therefore, widening is not required.

Structural Capacity: The Preconstruction Manual states, "If any existing structural member has a design capacity less than HS 15 (HS 20 for interstate bridges), replace that member." The Chena Hot Springs Road is not classified as an "interstate" highway and therefore, must satisfy HS 15 criteria. The bridge inventory ratings are greater than HS 20. Therefore, no strengthening is required.

Bridge Rail and Transitions: Inspection reports indicate the presence of missing, loose, and cut bridge rail anchor bolt nuts. A file review found that the 1990 rail retrofit design decreased the available nut installation width, which inadvertently required field modification of the anchorage system. The amount of field modification and affect on bridge rail performance cannot be calculated. Therefore, installation of new bridge rail and transitions are required.

Note: If the Alaska Multi-State Bridge Rail is used, then the usable bridge deck width will decrease from 28'-0" to 26'-7". This reduced usable bridge deck width exceeds the minimum required 3R width.

Earthquake Capacity: This bridge does not satisfy the 3R seismic bearing width criteria. Therefore, seismic retrofit, pier cap widening, is required.

Other: A purpose of 3R projects is to correct deficiencies identified by the Bridge Inspection Program and Maintenance and Operations. Recent inspections identify the following additional work items:

- Place backfill under Steese abutment cap to retain approach fill.
- Install Name Place and Object Marker signs at both bridge ends.

Cost Summary: The estimated rehabilitation construction costs follow:

Item No.	Item Description	Unit	Qty	Total Cost
205(4)	Porous Backfill	CY	10	$3,000
401(1)	Class A Concrete	CY	6	$14,000
501(10)	Coring Concrete	LF	75	$18,000
502(1)	Post Tensioning (CIP Concrete)	Each	28	$9,800
503(1)	Reinforcing Steel	LB	1,800	$6,000
504(1)	Structural Steel	LB	1,000	$7,500
507(1)	Steel Bridge Rail	LF	324	$64,800
606(12)	Guardrail/Bridge Rail Connection	EA	4	$8,000
615(1)	Standard Sign	SF	42	$3,500
640(1)	Mobilization and Demobilization	LS	(10%)	$13,500
	Contingencies		(15%)	$22,300
	Construction Engineering		(15%)	$22,300
TOTAL:				**$192,700**

Figure 23-1 — Sample Bridge 3R Memo
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Tiff Vincent 5 May 11, 2006

BRIDGE NAME	BRIDGE NUMBER	MILEPOINT	SUFFICIENCY RATING	LENGTH (Feet)	WIDTH (Feet)
North Fork Chena River	234	44.0	90.4	182.1	27.9

3R Criteria Evaluated

Width: The bridge has a design ADT of 247 and the usable bridge deck width is greater than the required 3R width. Based on this information, the existing bridge deck width satisfies the 3R criteria. Therefore, widening is not required.

Structural Capacity: The Preconstruction Manual states, "If any existing structural member has a design capacity less than HS 15 (HS 20 for interstate bridges), replace that member." The Chena Hot Springs Road is not classified as an "interstate" highway and therefore, must satisfy HS 15 criteria. The bridge inventory ratings are greater than HS 20. Therefore, no strengthening is required.

Bridge Rail and Transitions: Inspection reports indicate the presence of missing, loose, and cut bridge rail anchor bolt nuts. A file review found that the 1990 rail retrofit design decreased the available nut installation width, which inadvertently required field modification of the anchorage system. The amount of field modification and affect on bridge rail performance cannot be calculated. Therefore, installation of new bridge rail and transitions are required.

Note: If the Alaska Multi-State Bridge Rail is used, then the usable bridge deck width will decrease from 28'-0" to 26'-7". This reduced usable bridge deck width exceeds the minimum required 3R width.

Earthquake Capacity: This bridge does not satisfy the 3R seismic bearing width criteria. Therefore, seismic retrofit, pier cap widening, is required.

Other: A purpose of 3R projects is to correct deficiencies identified by the Bridge Inspection Program and Maintenance and Operations. Recent inspections identify the following additional work items:

- Install Name Place and Object Marker signs at both bridge ends.

Cost Summary: The estimated rehabilitation construction costs follow:

Item No.	Item Description	Unit	Qty	Total Cost
401(1)	Class A Concrete	CY	12	$28,000
501(10)	Coring Concrete	LF	150	$36,000
502(1)	Post Tensioning (CIP Concrete)	Each	56	$19,600
503(1)	Reinforcing Steel	LB	3,600	$12,000
504(1)	Structural Steel	LB	2,000	$15,000
507(1)	Steel Bridge Rail	LF	324	$72,800
606(12)	Guardrail/Bridge Rail Connection	EA	4	$8,000
615(1)	Standard Sign	SF	42	$3,500
640(1)	Mobilization and Demobilization	LS	(10%)	$19,500
	Contingencies		(15%)	$32,200
	Construction Engineering		(15%)	$32,200
TOTAL:				**$278,800**

Figure 23-1 — Sample Bridge 3R Memo
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BRIDGE NAME	BRIDGE NUMBER	MILEPOINT	SUFFICIENCY RATING	LENGTH (Feet)	WIDTH (Feet)
North Fork Chena River	235	45.7	90.4	122.0	27.9

3R Criteria Evaluated

Width: The bridge has a design ADT of 247 and the usable bridge deck width is greater than the required 3R width. Based on this information, the existing bridge deck width satisfies the 3R criteria. Therefore, widening is not required.

Structural Capacity: The Preconstruction Manual states, "If any existing structural member has a design capacity less than HS 15 (HS 20 for interstate bridges), replace that member." The Chena Hot Springs Road is not classified as an "interstate" highway and therefore, must satisfy HS 15 criteria. The bridge inventory ratings are greater than HS 20. Therefore, no strengthening is required.

Bridge Rail and Transitions: Inspection reports indicate the presence of missing, loose, and cut bridge rail anchor bolt nuts. A file review found that the 1990 rail retrofit design decreased the available nut installation width, which inadvertently required field modification of the anchorage system. The amount of field modification and affect on bridge rail performance cannot be calculated. Therefore, installation of new bridge rail and transitions are required.

Note: If the Alaska Multi-State Bridge Rail is used, then the usable bridge deck width will decrease from 28'-0" to 26'-7". This reduced usable bridge deck width exceeds the minimum required 3R width.

Earthquake Capacity: This bridge does not satisfy the 3R seismic bearing width criteria. Therefore, seismic retrofit, pier cap widening, is required.

Other: A purpose of 3R projects is to correct deficiencies identified by the Bridge Inspection Program and Maintenance and Operations. Recent inspections identify the following additional work items:

- Install Name Place and Object Marker signs at both bridge ends.

Cost Summary: The estimated rehabilitation construction costs follow:

Item No.	Item Description	Unit	Qty	Total Cost
401(1)	Class A Concrete	CY	6	$14,000
501(10)	Coring Concrete	LF	75	$18,000
502(1)	Post Tensioning (CIP Concrete)	Each	28	$9,800
503(1)	Reinforcing Steel	LB	1,800	$6,000
504(1)	Structural Steel	LB	1,000	$7,500
507(1)	Steel Bridge Rail	LF	244	$48,800
606(12)	Guardrail/Bridge Rail Connection	EA	4	$8,000
615(1)	Standard Sign	SF	42	$3,500
640(1)	Mobilization and Demobilization	LS	(10%)	$11,600
	Contingencies		(15%)	$19,100
	Construction Engineering		(15%)	$19,100
TOTAL:				**$165,400**

Figure 23-1 — Sample Bridge 3R Memo
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BRIDGE NAME	BRIDGE NUMBER	MILEPOINT	SUFFICIENCY RATING	LENGTH (Feet)	WIDTH (Feet)
North Fork Chena River	237	48.9	90.4	182.1	27.9

3R Criteria Evaluated

Width: The bridge has a design ADT of 247 and the usable bridge deck width is greater than the required 3R width. Based on this information, the existing bridge deck width satisfies the 3R criteria. Therefore, widening is not required.

Structural Capacity: The Preconstruction Manual states, "If any existing structural member has a design capacity less than HS 15 (HS 20 for interstate bridges), replace that member." The Chena Hot Springs Road is not classified as an "interstate" highway and therefore, must satisfy HS 15 criteria. The bridge inventory ratings are greater than HS 20. Therefore, no strengthening is required.

Bridge Rail and Transitions: Inspection reports indicate the presence of missing, loose, and cut bridge rail anchor bolt nuts. A file review found that the 1990 rail retrofit design decreased the available nut installation width, which inadvertently required field modification of the anchorage system. The amount of field modification and affect on bridge rail performance cannot be calculated. Therefore, installation of new bridge rail and transitions are required.

Note: If the Alaska Multi-State Bridge Rail is used, then the usable bridge deck width will decrease from 28'-0" to 26'-7". This reduced usable bridge deck width exceeds the minimum required 3R width.

Earthquake Capacity: This bridge does not satisfy the 3R seismic bearing width criteria. Therefore, seismic retrofit, pier cap widening, is required.

Other: A purpose of 3R projects is to correct deficiencies identified by the Bridge Inspection Program and Maintenance and Operations. Recent inspections identify the following additional work items:

- Install Name Place and Object Marker signs at both bridge ends.

Cost Summary: The estimated rehabilitation construction costs follow:

Item No.	Item Description	Unit	Qty	Total Cost
401(1)	Class A Concrete	CY	12	$28,000
501(10)	Coring Concrete	LF	150	$36,000
502(1)	Post Tensioning (CIP Concrete)	Each	56	$19,600
503(1)	Reinforcing Steel	LB	3,600	$12,000
504(1)	Structural Steel	LB	2,000	$15,000
507(1)	Steel Bridge Rail	LF	364	$72,800
606(12)	Guardrail/Bridge Rail Connection	EA	4	$8,000
615(1)	Standard Sign	SF	42	$3,500
640(1)	Mobilization and Demobilization	LS	(10%)	$19,500
	Contingencies		(15%)	$32,200
	Construction Engineering		(15%)	$32,200
TOTAL:				**$278,800**

Figure 23-1 — Sample Bridge 3R Memo
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BRIDGE NAME	BRIDGE NUMBER	MILEPOINT	SUFFICIENCY RATING	LENGTH (Feet)	WIDTH (Feet)
Angel Creek	238	49.9	91.2	82.0	27.9

3R Criteria Evaluated

Width: The bridge has a design ADT of 247 and the usable bridge deck width is greater than the required 3R width. Based on this information, the existing bridge deck width satisfies the 3R criteria. Therefore, widening is not required.

Structural Capacity: The Preconstruction Manual states, "If any existing structural member has a design capacity less than HS 15 (HS 20 for interstate bridges), replace that member." The Chena Hot Springs Road is not classified as an "interstate" highway and therefore, must satisfy HS 15 criteria. The bridge inventory ratings are greater than HS 20. Therefore, no strengthening is required.

Bridge Rail and Transitions: Inspection reports indicate the presence of missing, loose, and cut bridge rail anchor bolt nuts. A file review found that the 1990 rail retrofit design decreased the available nut installation width, which inadvertently required field modification of the anchorage system. The amount of field modification and affect on bridge rail performance cannot be calculated. Therefore, installation of new bridge rail and transitions are required.

Note: If the Alaska Multi-State Bridge Rail is used, then the usable bridge deck width will decrease from 28'-0" to 26'-7". This reduced usable bridge deck width exceeds the minimum required 3R width.

Earthquake Capacity: This bridge satisfies the 3R seismic criteria. Therefore, no seismic retrofit is required.

Other: A purpose of 3R projects is to correct deficiencies identified by the Bridge Inspection Program and Maintenance and Operations. Recent inspections identify the following additional work items:

- Install Name Place and Object Marker signs at both bridge ends.

Cost Summary: The estimated rehabilitation construction costs follow:

Item No.	Item Description	Unit	Qty	Total Cost
507(1)	Steel Bridge Rail	LF	164	$32,800
606(12)	Guardrail/Bridge Rail Connection	EA	4	$8,000
615(1)	Standard Sign	SF	42	$3,500
640(1)	Mobilization and Demobilization	LS	(10%)	$4,500
	Contingencies		(15%)	$7,400
	Construction Engineering		(15%)	$7,400
TOTAL:				**$63,600**

Figure 23-1 — Sample Bridge 3R Memo
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BRIDGE NAME	BRIDGE NUMBER	MILEPOINT	SUFFICIENCY RATING	LENGTH (Feet)	WIDTH (Feet)
West Fork Chena River	239	52.4	91.2	163.1	27.9

3R Criteria Evaluated

Width: The bridge has a design ADT of 247 and the usable bridge deck width is greater than the required 3R width. Based on this information, the existing bridge deck width satisfies the 3R criteria. Therefore, widening is not required.

Structural Capacity: The Preconstruction Manual states, "If any existing structural member has a design capacity less than HS 15 (HS 20 for interstate bridges), replace that member." The Chena Hot Springs Road is not classified as an "interstate" highway and therefore, must satisfy HS 15 criteria. The bridge inventory ratings are greater than HS 20. Therefore, no strengthening is required.

Bridge Rail and Transitions: Inspection reports indicate the presence of missing, loose, and cut bridge rail anchor bolt nuts. A file review found that the 1990 rail retrofit design decreased the available nut installation width, which inadvertently required field modification of the anchorage system. The amount of field modification and affect on bridge rail performance cannot be calculated. Therefore, installation of new bridge rail and transitions are required.

Note: If the Alaska Multi-State Bridge Rail is used, then the usable bridge deck width will decrease from 28'-0" to 26'-7". This reduced usable bridge deck width exceeds the minimum required 3R width.

Earthquake Capacity: This bridge does not satisfy the 3R seismic bearing width criteria. Therefore, seismic retrofit, pier cap widening, is required.

Other: A purpose of 3R projects is to correct deficiencies identified by the Bridge Inspection Program and Maintenance and Operations. Recent inspections identify the following additional work items:

- Place backfill under Chena Hot Springs abutment cap to retain approach fill.
- Install Name Place and Object Marker signs at both bridge ends.

Cost Summary: The estimated rehabilitation construction costs follow:

Item No.	Item Description	Unit	Qty	Total Cost
205(4)	Porous Backfill	CY	10	$3,000
401(1)	Class A Concrete	CY	6	$14,000
501(10)	Coring Concrete	LF	75	$18,000
502(1)	Post Tensioning (CIP Concrete)	Each	28	$9,800
503(1)	Reinforcing Steel	LB	1,800	$6,000
504(1)	Structural Steel	LB	1,000	$7,500
507(1)	Steel Bridge Rail	LF	324	$64,800
606(12)	Guardrail/Bridge Rail Connection	EA	4	$8,000
615(1)	Standard Sign	SF	42	$3,500
640(1)	Mobilization and Demobilization	LS	(10%)	$13,500
	Contingencies		(15%)	$22,300
	Construction Engineering		(15%)	$22,300
TOTAL:				**$192,700**

Figure 23-1 — Sample Bridge 3R Memo
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1. *FHWA Workshop*, “Rehabilitation of Existing Bridges,” 1984 (Revised 1986)
2. *NCHRP Report 206*, “Detection and Repair of Fatigue Damage in Welded Highway Bridges,” 1978
3. *FHWA-RD-78-133*, “Extending the Service Life of Existing Bridges by Increasing Their Load-Carrying Capacity,” 1978
4. *NCHRP Report 222*, “Bridges on Secondary Highways and Local Roads — Rehabilitation and Replacement,” 1980
5. *NCHRP Report 226*, “Damage Evaluation and Repair Methods for Prestressed Concrete Bridge Members,” 1980
6. *NCHRP Project 12-17 Final Report*, “Evaluation of Repair Techniques for Damaged Steel Bridge Members: Phase I,” 1981
7. *NCHRP Report 243*, “Rehabilitation and Replacement of Bridges on Secondary Highways and Local Roads,” 1981
8. *NCHRP Report 244*, “Concrete Sealers for Protection of Bridge Structures,” 1981
9. *FHWA-RD-82-041*, “Innovative Methods of Upgrading Deficient Through Truss Bridges,” 1982
10. *FHWA-RD-83-007*, “Seismic Retrofitting Guidelines for Highway Bridges,” 1983
11. *NCHRP Report 271*, “Guidelines for Evaluation and Repair of Damaged Steel Bridge Members,” 1984
12. *NCHRP Report 280*, “Guidelines for Evaluation and Repair of Prestressed Concrete Bridge Members,” 1985
13. *NCHRP Synthesis of Highway Practice 119*, “Prefabricated Bridge Elements and Systems,” 1985
14. *NCHRP Report 293*, “Methods of Strengthening Existing Highway Bridges,” 1987
15. *NCHRP Report 297*, “Evaluation of Bridge Deck Protective Strategies,” 1987
16. *NCHRP Report 312*, “Condition Surveys of Concrete Bridge Components,” 1988
17. *NCHRP Report 321*, “Welded Repair of Cracks in Steel Bridge Members,” 1989
18. *NCHRP Report 655,* “Recommended Guide Specification for the Design of Externally Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements,” 2010
19. *NCHRP Synthesis of Highway Practice 398*, “Cathodic Protection for Life Extension of Existing Reinforced Concrete Bridge Elements,” 2009
20. *NCHRP Report 604*, “Heat-Straightening Repair of Damaged Steel Bridge Girders: Fatigue and Fracture Performance,” 2008
21. *Task Force 30 Report*, “Guide Specifications for Concrete Overlay of Pavements and Bridge Decks,” 1990
22. *SHRP-S-344* “Rapid Concrete Bridge Deck Protection, Repair and Rehabilitation,” 1993
23. *SHRP-S-360*, “Concrete Bridge Protection, Repair and Rehabilitation — A Methods Application Manual,” 1993
24. National Park Service “The Secretary of the Interior’s Standards for Rehabilitation,” 1992
25. National Park Service “The Secretary of the Interior’s Standards for the Treatment of Historic Properties,” 1992

Figure 23-2
Bridge Rehabilitation Literature

23.3. Bridge Condition Surveys and Tests

This section presents DOT&PF policies and practices for condition surveys and tests for a bridge rehabilitation project.

The discussion does *not* pertain to any condition surveys and tests performed for the Alaska Bridge Inspection Program (see Chapter 26) nor the DOT&PF Bridge Management System (see Chapter 28).

23.3.1. General

The bridge engineer is responsible for arranging and conducting field surveys, requesting specific tests performed by others (e.g., Statewide Materials), and evaluating data collected during the field survey and provided by others.

23.3.2. Concrete Bridge Decks

This section applies to deck-on-girder bridges. Concrete bridge decks include the structural continuum directly supporting the riding surface plus expansion joints, curbs, barriers, approach slabs, and utility hardware (if suspended from the deck). The bridge deck and its appurtenances:

- support and distribute wheel loads to the primary structural components;
- protect the structural components beneath the deck;
- provide a smooth riding surface; and
- provide a safe passageway for vehicular and bicycle/pedestrian traffic (e.g., skid-resistant surface, bridge rails, guardrail-to-bridge-rail transitions).

Any deterioration in these functions warrants investigation and possible remedial action.

The most common source of concrete bridge deck deterioration is the intrusion of chloride ions from roadway deicing agents into the concrete. The chloride causes formation of corrosive cells on the steel reinforcement, and the increased volume of the corrosion product (rust) induces stresses in the concrete resulting in cracking, delamination, and spalling.

Chloride ion (salt) penetration is a time-dependent phenomenon. There is no proven way to absolutely prevent penetration, but it can be slowed such that the service life of the deck is not less than that of the remaining structure.

Chloride penetration is, however, not the only cause of bridge deck deterioration. Other significant problems include:

- **Freeze-Thaw**. Results from inadequate air content of the concrete. Freezing of the free water in the concrete causes random, alligator cracking of the concrete and then complete disintegration. There is no known remedy other than replacement.
- **Impact Loading**. Results from vehicular kinetic energy released by vertical discontinuities in the riding surface, such as surface roughness, delamination, and inadequately set or damaged expansion joints. Remedial actions are surface grinding, overlay or replacement of deck concrete, and rebuilding expansion joints.
- **Abrasion**. Normally results from metallic objects, such as snow plow blades, chains, or studded tire wear. Remedial actions are surface grinding or overlay.

Certain factors are symptomatic indicators that a bridge deck may have a shorter than expected service life or that it is actually in the latter phases of its service life. Some examples are:

- extensive cracking (shrinkage, stress, etc.);
- extensive delamination;
- exposed reinforcing steel; and
- spalls.

The deck can be placed into one of the following categories (based on NBI ratings):

- Very good decks that need little attention. These are the (8) and (9) rated decks.
- Decks that are in reasonably good shape and need no substantial repair, but nominal maintenance expenditure can extend deck life. These are the (7) rated decks. Decks in this condition range most likely need some minor crack sealing and/or minor patching.
- Decks that need considerable repair but are still quite sound and capable of serving adequately for five to ten more years. These are candidates for repair and overlay with some type of non-permeable concrete. These are the (5) and (6)

rated decks. The bridge engineer would most likely consider an overlay for bridge decks in this condition range, depending on the extent of chloride contamination.

- Decks that are no longer serviceable and will soon need replacement regardless of any remedial action. Significant expenditures of funds are not justified until replacement. These are the (3) and (4) rated decks. Decks in these conditions fall into the "replace or rehabilitate deck" category, with depth of rehabilitation determined by the chloride content. Typically, the as-built concrete cover over the top mat of rebar is not well known and care must be taken in removing this concrete.

When considering a bridge for deck rehabilitation, the Bridge Section may request a number of tests to collect data on the deck condition. The data allows the bridge engineer to determine whether deck rehabilitation or deck replacement is appropriate and, if the choice is rehabilitation, the information allows the determination of the appropriate type of treatment.

A deck evaluation may include gathering the following information:

- a plot locating existing delaminations, spalls, and cracks;
- representative measurements of crack width;
- measurements of the depth of cover to the top mat of reinforcing steel;
- sampling and laboratory analysis to determine the existing levels of chloride contamination;
- measurements of electrical potential on a grid pattern to locate areas of active corrosion; and
- deck concrete compressive strength assessed through destructive testing of deck core samples.

Expect to obtain at least some degree of confirmation and conflicting test results because these field tests each have a degree of uncertainty. Thus, sampling multiple locations within a traffic lane is important to estimate the true condition of the deck and the extent of active corrosion.

Apply engineering judgment when analyzing multiple test results. The following provides more information on each type of data collected and their use in determining an appropriate deck treatment.

Visual Inspection

Description: A visual inspection of the bridge deck should establish:

- the approximate extent of cracking, representative crack widths, and spalling;
- evidence of any corrosion;
- evidence of pattern cracking, efflorescence, or dampness on the deck underside;
- rutting of the riding surface and/or ponding of water;
- performance of expansion joints;
- functionality of deck drainage system; and
- bridge rails and guardrail-to-bridge-rail transitions meeting current DOT&PF standards.

Purpose: Visual inspection of the bridge deck will accomplish the following:

- By establishing the approximate extent of cracking and crack width, corrosion, delamination, and spalling (and by having evidence of other deterioration), the bridge engineer can determine if a more extensive inspection is warranted.

- The inspection will identify substandard roadside safety appurtenances.

When to Use: On all potential deck rehabilitation projects.

Analysis of Data: Pattern cracking, heavy efflorescence, or dampness on the deck underside suggests that this portion of the deck is likely to be highly contaminated and active corrosion is taking place. In addition, the bridge engineer should consider:

- traffic control that will be required,
- timing of repair,
- age of structure,
- average annual daily traffic (AADT),
- slab depth,
- structure type, and
- depth of cover to reinforcement.

Delamination Sounding

Description: Establishes the presence of delamination, based on audible observation, by chain drag or hammer. Based on the observation that delaminated concrete responds with a "hollow sound" when struck by a metal object. See ASTM D4580 "Standard Practice for Measuring Delaminations in

Concrete Bridge Decks by Sounding." See Appendix 26.A for more discussion.

Purpose: To determine the location and area of delamination.

When to Use: On all concrete deck rehabilitation projects, except where asphalt overlays prevent performance of the test.

Analysis of Data: Decisions are made on a case-by-case basis, but the following recommendations can assist decision-making based on the extent of the bridge deck spalling:

- Consider remedial action when 10 percent of the surface area is delaminated.
- Consider bridge deck rehabilitation when 40 percent of the surface is delaminated.

Chloride Analysis

Description: A chemical analysis of pulverized samples of concrete extracted from the bridge deck. Determines concentration of water-soluble chlorides by using the *Gravimetric Method —Silver Chloride Method* as described in *Scott's Standard Methods of Chemical Analysis*, 6th Edition, March 1962, (D. Van Nostrand).

As an option, chloride testing may be conducted using potentiometric titration with silver nitrate per AASHTO T 260 *Sampling and Testing for Chloride Ion in Concrete and Concrete Raw Materials*. See Appendix 26.A for more discussion.

Purpose: To determine the chloride content profile from the deck surface to the top mat of rebar.

When to Use: Use on bridge decks where the need for major rehabilitation or replacement is anticipated. Take chloride samples at three to five locations along the travel lane per span from each span 100 feet or less in length. Increase the number of samples for longer spans.

Analysis of Data: The "threshold" or minimum level of water-soluble chloride contamination in concrete necessary to corrode reinforcing steel is approximately 255 ppm or 1.2 lbs of chloride per cubic yard in the concrete down to the top mat of rebar. Chloride concentrations of less than this threshold indicate a sound deck that will in most cases not require rehabilitation. Consider adding a deck protection system.

Chloride concentrations within or greater than this range above the top reinforcing mat require the removal of at least enough concrete so that the remaining concrete contamination is below the threshold.

Threshold or greater chloride concentrations at the level of the top reinforcing mat require either demolition removing enough concrete to ensure that the remaining concrete is below the threshold values, or possibly deck replacement. Threshold contamination or worse at or near the level of the bottom mat of reinforcing steel may require deck replacement.

Coring

Description: Take 2-inch or 4-inch diameter cylindrical cores. Avoid cutting into the rebar. In decks with large amounts of reinforcement, it is difficult to avoid cutting steel if 4-inch diameter cores are used.

Purpose: To establish strength, composition of concrete, crack depth, position of reinforcing steel.

When to Use: On all concrete deck rehabilitation projects when doubt exists on the compressive strength or soundness of the concrete or if the visual condition of the reinforcement is desired.

Analysis of Data: Less than 2 inches of concrete cover is inadequate for corrosion protection. If compressive strengths are less than 3 ksi, the bridge engineer must determine whether to proceed with the deck rehabilitation or to proceed with a deck replacement.

Pachometer Readings

Description: The pachometer produces a magnetic field in the bridge deck and displays a disruption in the magnetic field (e.g., induced by a steel reinforcing bar).

Purpose: To determine the location and depth of steel reinforcing bars. Establish these properties to a depth of approximately 4 inches.

When to Use: Use pachometer readings on all concrete rehabilitation projects to verify reinforcement location as needed. Use to locate steel to avoid damage when drilling or coring concrete.

Pull-Off Test

Description: The pull-off test determines the perpendicular tensile force that a surface area can resist before a plug of material is detached from the concrete deck per ASTM D7234 – 05 “Standard Test Method for Pull-Off Adhesion Strength of Coatings on Concrete Using Portable Pull-Off Adhesion Testers.” Failure occurs along the weakest plane within the system comprised of the overlay and concrete deck. Clean the surface of the deck to obtain a dry deck.

Purpose: Determines the soundness of concrete to successfully receive an overlay.

When to Use: Use pull-off tests when an overlay is applied to a concrete deck of questionable soundness.

23.3.3. Superstructure

As defined in this *Manual*, the superstructure consists of the bearings and all of the components and elements resting upon them. The following briefly describes those condition surveys and tests that may be performed on the superstructure elements (other than bridge decks) to determine the appropriate level of rehabilitation.

Visual Inspection

Description: A visual inspection of the superstructure should include an investigation of the following to supplement the information contained in the NBI bridge inspection report:

- surface deterioration, cracking, and spalling of concrete
- major loss in concrete components
- evidence of efflorescence
- corrosion of reinforcing steel or prestressing tendons
- section loss in exposed reinforcing steel or prestressing tendons
- peeling coating system
- corrosion of structural metal components
- section loss in metal components due to corrosion
- cracks in metal components
- measurement of deformed shapes
- loose or missing rivets or bolts
- deterioration and loss in wood components
- collision damage by vehicles, vessels, or debris
- leaking expansion joints
- ponding of water on abutment seats
- measurement of deformed shapes, which can contribute to moment magnification
- condition and functionality of bearings
- distress in pedestals and bearing seats

Purpose: To record all deterioration and signs of potential distress for comparison with earlier records and for initiating rehabilitation procedures if warranted. This is the primary method used to assess the condition of the superstructure.

When to Use: On all bridge rehabilitation projects.

Analysis of Data: As required, if the deterioration is deemed significant enough to result in loss of load-carrying capacity.

Fracture-Critical Members (Steel)

Description: Fracture-critical members or member components (FCMs) are steel tension members or steel tension components of members whose failure would likely result in a partial or full collapse of the bridge. The Bridge Section has identified all fracture-critical structures in Alaska. The bridge engineer must recognize typical fracture-critical details when conducting the field review because it may affect the scope of bridge rehabilitation. Typical bridges in Alaska containing fracture-critical members are:

- steel trusses (pins, eye-bars, bottom chords, other tension members);
- two-girder steel bridges;
- transverse girders (supporting longitudinal beams and girders); and
- pin-and-hanger connections (located on suspended spans or at transverse girders).

Purpose: To identify FCMs in the project. Distress of FCMs requires special consideration.

When to Use: On all bridge rehabilitation projects.

Analysis of Data: As required, if the deterioration is deemed significant enough to result in loss of load-carrying capacity.

Cracking in Steel

Description: If visual inspection by the bridge inspector reveals cracking in steel components, establish the extent and size of cracks to determine the appropriate remedial action. The following are the most common test methods performed by DOT&PF to locate cracks in steel components and measure their extent and size:

1. **Dye-Penetrant Testing (PT)**. This is the primary method used. Clean and paint the surface of the steel with a red dye. Allow time for the dye to “dwell” on the area and then wipe off. If a crack is present, the dye penetrates the

crack through capillary action. To indicate where the red dye "bleeds" from the crack, paint a white developer on the cleaned steel.

2. **Magnetic-Particle Testing (MT)**. Clean and sprinkle the surface of the steel with fine iron filings while a strong magnetic field is induced in the steel. A crack causes an interruption in the lines of magnetic flux, allowing them to "leak" from the metal, thereby attracting the metal filings, which form a trace along the line of the crack.

3. **Ultrasonic Testing (UT).** Testing devices that use high-frequency sound waves to detect cracks, discontinuities, and flaws in materials. The accuracy of UT depends upon the expertise of the individual conducting the test and interpreting the results.

4. **Eddy Current Testing (ET).** Eddy current testing uses the phenomenon of electromagnetic induction to detect flaws in conductive materials. This form of testing detects flux leakage emanating from a discontinuity in metal when an eddy current passes through the material. Eddy current testing can detect very small cracks in or near the surface of the material, the surfaces need minimal preparation, and physically complex geometries can be quickly investigated.

5. **Radiographic Testing (RT).** Radiographic testing uses X-rays, produced by an X-ray tube, or gamma rays, produced by a radioactive isotope. The basic principle of radiographic inspection of welds is the same as that for medical radiography. Penetrating radiation is passed through a solid object, in this case a weld rather that part of the human body, onto a photographic film, resulting in an image of the object's internal structure being deposited on the film. The amount of energy absorbed by the object depends on its thickness and density. Energy not absorbed by the object will cause exposure of the radiographic film. These areas will be dark when the film is developed. Areas of the film exposed to less energy remain lighter. Therefore, areas of the object where the thickness has been changed by discontinuities, such as porosity or cracks, will appear as dark outlines on the film. Inclusions of low density, such as slag, will appear as dark areas on the film while inclusions of high density, such as tungsten, will appear as light areas. All discontinuities are detected by viewing shape and variation in density of the processed film.

At a minimum, a Level II ASNT certified technician must conduct all tests. For more information, see *Detection and Repair of Fatigue Damage in Welded Highway Bridges*, NCHRP Report 206, July 1979.

Purpose: To quantify the extent of fatigue cracking,

When to Use: In rehabilitation projects where fatigue cracks are identified.

Analysis of Data: As required for each specific testing technique.

Load-Induced Fatigue Analysis (Steel)

Description: Fatigue is defined as steady-state crack growth. Failure of the component can result from growth of existing flaws in steel members to a critical size at which fracture is no longer effectively resisted by the toughness of the steel. The crack growth is a function of:

- crack size;
- location of crack (i.e., stress concentration at the structural detail);
- toughness (energy-absorbing characteristics of metal);
- temperature; and
- frequency and level of nominal stress range (transient stresses).

Purpose: To establish type and urgency of remedial action.

When to Use: Where cracks, found by visual inspection, are believed to be caused by fatigue or at fatigue-prone details.

Analysis of Data: A structural engineer experienced in fatigue-life assessment should perform the analysis. Establish fatigue characteristics of the metal for the analysis. For the stress range, the *LRFD Specifications* provide an upper-bound criterion of 75 percent weight of one design truck plus impact per bridge. The actual stress range of a given bridge component may be far lower than that specified by the *LRFD Specifications*, and it may be warranted to establish it by physical means.

Load Testing (Steel and Concrete)

Description: Measuring strains in girders can capture the actual live-load distribution. In most cases, these measured strains suggest an enhanced distribution compared to the empirical values in the applicable AASHTO bridge specifications.

Purpose: To determine more favorable load ratings.

When to Use: Where unacceptable load ratings result from the empirical distribution equations. The DOT&PF bridge instrumentation program can be used to quantify the live-load distribution for use in load ratings.

Sounding and Penetration Tests (Timber Bridges)

Description: Sounding tests using a hammer and/or penetration tests using an ice pick. See Section 26.5.5, Special Bridge Inspection Practices, and Appendix 26.A for optional equipment and tests.

Purpose: To establish the soundness of wood components.

When to Use: Where the soundness of wood components is in question.

23.3.4. Substructures and Foundations

Visual Inspection

Description: A visual inspection of the substructure components should address the following to supplement the NBI bridge inspection report:

- surface deterioration, cracking, and spalling of concrete
- major section loss in concrete components
- evidence of corrosion in reinforcing steel
- section loss in exposed reinforcing steel
- deterioration or loss of integrity in wood components
- leaking joints and cracks
- deformations contributing to moment magnification
- collision damage
- changes in geometry such as settlement, rotation of wingwalls, tilt of retaining walls, etc.
- seismic vulnerabilities
- accumulation of debris
- erosion of protective covers
- changes in embankment and water channel
- evidence of significant scour

Purpose: To record all deterioration and signs of potential distress for comparison with earlier records and for initiating rehabilitation procedures if warranted.

When to Use: On all potential bridge rehabilitation projects.

Analysis of Data: As required, if the deterioration is deemed significant enough to result in loss of load-carrying capacity.

Delamination Sounding

The relevant superstructure criteria apply to substructures. See Section 23.3.2.

Soundings and Penetration Tests (Timber Bridges)

The relevant superstructure criteria apply to substructures. See Section 23.3.3.

FCM Pier Caps with Steel

The relevant superstructure criteria apply to substructures. See Section 23.3.3.

23.3.5. Special Provisions

If a special provision is required for a bridge rehabilitation project, address some of the key issues including:

- surface preparation,
- materials specifications,
- types of equipment,
- bonding methods/specifications,
- curing,
- protection of adjacent bridge elements,
- repair of damage,
- weather limitations,
- required testing after construction, and
- opening to traffic.

23.4. Bridge Deck Rehabilitation

23.4.1. General

Chapter 16 provides an in-depth discussion on the design of bridge decks that are constructed compositely in conjunction with concrete and steel girders for new bridges. Many of the Chapter 16 design and detailing practices may also apply to deck rehabilitation.

23.4.2. Patching

A permanent repair can be assured only if all concrete is removed in areas having a chloride content sufficient to sustain corrosion.

For partial depth repairs, remove concrete to a depth of ¼ inch plus the maximum size of the aggregate below the bottom of the top mat of reinforcing steel. The actual corrosion threshold can be as low as 1.3 lb of Cl per cubic yard of a typical deck concrete, but a value of 2 lb of Cl per cubic yard is commonly accepted as the level beyond which removal of the concrete is warranted. Unless the contaminated concrete is removed, differences in the surface conditions on the reinforcing bar may cause the formation of anodic and cathodic areas and a resumption of the corrosion process. However, removal of concrete below the reinforcing steel may be extremely costly, and complete removal and replacement of the deck may be more economical.

An evaluation of the corrosion process indicates that patches cannot be considered permanent repairs, and field experience tends to verify this conclusion. Newly delaminated areas are often found adjacent to areas patched months before. Nevertheless, patching can be an appropriate temporary action until more extensive restoration is performed, and it can provide substantial service with the subsequent installation of a protective overlay.

A wide variety of materials have been used for patching bridge decks. Although conventional Portland cement concrete is often used, many other materials are available to provide rapid strength development and to allow early opening of the deck to traffic. It is essential to follow the manufacturer's requirements for mixing, placing, and curing. A polymer concrete overlay, if used, can also serve as the deck patching material.

Bonding components vary with the repair materials. Some prepackaged polymer-modified concretes develop sufficient adherence so that a bonding agent is not required. Consult the manufacturers of all prepackaged, fast-setting patching materials for the proper bonding agents. A methacrylate primer is used for polymer overlay patches.

23.4.3. Crack Repair

Epoxy-Resin Injection

Epoxy-resin injection is commonly used to fill cracks in decks. Because the resin is injected under pressure, it is usually possible to fill the entire depth of crack.

Methacrylate Sealant

A low-viscosity organic liquid compound is flooded over the deck, and fills the cracks by gravity and capillary action. Accordingly, the success of this operation depends on the crack size, selection of the appropriate compound, temperature, contamination on the crack walls, and the skill of the operator. The contractor must clean the deck surface prior to application of the sealant.

23.4.4. Waterproof Membrane/Asphalt Overlay

DOT&PF typically uses waterproof membranes with an asphalt overlay on new decked bulb-tee girder bridges, and the Department has experienced good results with their use. On deck-on-girder bridges, a waterproof membrane with asphalt overlay can also demonstrate better performance than concrete or polymer overlays for deck rehabilitation.

A waterproof membrane with asphalt overlay has comparable construction time frames as the other overlay systems. The surface preparation for the membrane is minimal. Only high points or exposed rocks must be removed so that they will not puncture the membrane.

Review bridges on highways programmed for pavement overlays to ensure safety, load capacity, and performance are not adversely affected. Consider the factors below in determining an acceptable overlay strategy.

Overlay Impact on Live Load Capacity

Review plans to determine what wearing surface dead load (asphalt thickness) the bridge was designed to accommodate.

Verify that the current load rating uses the proposed asphalt thickness. If not, recalculate load rating with proposed asphalt thickness.

On the NHS, inventory load ratings should not decrease below HS 25. Off the NHS, inventory load ratings should not decrease below HS20. If necessary, limit asphalt thickness to achieve these values.

Verify that the proposed asphalt thickness does not decrease the operating load rating for the route on which the bridge is located. If necessary, limit the asphalt thickness so that the route's operating rating does not decrease.

Overlay Impact on Bridge Rail Height

Verify that the proposed asphalt thickness does not decrease the rail height below the following:

1. For crash tested bridge railing, 1-inch plus or minus the crash tested height.
2. For non-crash tested bridge railing, 27 inch minimum height.

Overlay Impact on Bridge Deck Joints

If bridge deck joints exist, project options are:

1. Match existing expansion joint height. In some situations this may require tapering the asphalt thickness.
2. Adjust the expansion joint height to match the proposed asphalt thickness.

Need for a Waterproof Membrane

Place a waterproof membrane below asphalt overlays.

When the existing asphalt thickness allows the top wearing surface to be milled, review bridge inspection reports to determine if the deck is currently watertight.

If there is evidence of water actively leaking through the bridge deck, consider removing all asphalt and installing a new waterproof membrane.

23.4.5. Microsilica Concrete Overlay

Microsilica concrete overlays provide a durable, smooth and economical riding surface that is resistant to chloride penetration and delamination.

Microsilica is a pozzolanic material that is much finer than cement particles, which allows it to produce a denser matrix. One of the biggest advantages of a microsilica concrete overlay is its reduced permeability to chloride penetration. Compressive strength is enhanced as well.

Although microsilica concrete is highly resistant to chloride penetration, the permeability of the microsilica concrete is also highly dependent on the quality of construction and proper curing of the deck.

23.4.6. Polyester Concrete Overlay

Polyester concrete is a composite of dry aggregate in an unsaturated, or thermoset, polyester resin binder. When the liquid resin cures into a hardened, cross-linked state, this forms a polyester concrete. The primary features and resulting benefits are:

- ease of application for reduced production costs,
- quicker cure for shorter lane closure times,
- thinner overlays for greater live load capacity,
- higher elongation and tensile strength for improved dynamic performance,
- a protective barrier against moisture and deicers for lower maintenance costs and longer service life, and
- greater resistance to abrasion and impact for lower maintenance costs and longer service life.

23.4.7. Cathodic Protection

The advantage of cathodic protection is that it can halt the progress of corrosion without the removal of chloride-contaminated concrete. Corrosion requires an anode, a point on the reinforcing steel where ions are released.

Cathodic protection can be either passive or active. Passive protection uses an anode that is more electrochemically active than the reinforcing steel. Active protection uses an anode plus an impressed current to increase the current flow. Anodes vary in shapes, sizes and metals.

Cathodic protection is seldom used because of several disadvantages, including:

- need for expertise in design and construction,
- need for periodic adjustment, and
- power requirement.

For more information, see *Guide Specifications for Cathodic Protection of Concrete Bridge Decks*, 1994, AASHTO Task Force 29.

23.4.8. Joint Rehabilitation and Replacement

Joint rehabilitation refers to the repair of a portion of an existing joint and not complete replacement. Joint rehabilitation includes repairing or replacing loose or broken restrainers on strip seal expansion joints, failed header materials adjacent to joints or torn seals. In most cases, the failure is due to vehicle impact. Failure may also be due to incompressible materials in the joint.

Bridges with asphalt overlays require a concrete header adjacent to the expansion joint. Concrete headers should be at least 8 inches but preferably 12 inches wide. Deck concrete should be removed down to a distance below the top mat of reinforcing steel to provide development length for the new header reinforcement.

Use a minimum number of joint splices with a full-length seal preferred. Torn strip seals can be repaired by vulcanizing or gluing; however, vulcanizing is preferred.

Where joint rehabilitation is not feasible, a replacement of an existing damaged or malfunctioning joint may be necessary. Chapter 19 provides guidance on joint selection and design.

23.4.9. Upgrade/Retrofit Bridge Rails/Approach Rails

Evaluation

Section 16.5.1 presents DOT&PF practices for new bridge rails. Ideally, existing bridge rails on a bridge rehabilitation project will meet the criteria in Section 16.5.1 or will be replaced with a new, NCHRP-350 compliant bridge rail. However, this is not always practical for a variety of reasons (e.g., dead load considerations, incompatibility with an existing bridge deck).

Table 23-1 presents the requirements for addressing bridge rails on existing bridges. This table applies to the maintenance and rehabilitation of existing bridges, including existing bridges within the limits of 3R roadway projects.

In addition to Table 23-1, examine the following when evaluating an existing bridge rail:

1. **Critical Design Details**. Inspect the existing bridge rail to verify the integrity of critical design details, such as:

   a. base plate connections,
   b. anchor bolts,
   c. welding details,
   d. concrete cracking, and
   e. reinforcement development.

2. **Safety Deficiencies**. Review the maintenance and repair history of the bridge rail. Even with no history of impact damage, an inspection of the existing bridge rail may reveal inherent safety deficiencies in the rail design, such as:

   a. potential for snagging (i.e., no blockouts)
   b. presence of curb in front of bridge rail
   c. inadequate height
   d. inadequate guardrail-to-bridge-rail transition

23.4.10. Bridge Deck Strengthening

A deck is strengthened by either reinforcing the decking material or decreasing the distance between the deck supports. Strengthening a concrete deck involves adding a system of floorbeams and stringers between the existing longitudinal beams.

Strengthening any deck involves the addition of weight, which reduces the live-load capacity of the supporting members. A structurally deficient deck due to insufficient steel requires strengthening. Use structural analysis to determine the size and spacing of supports to provide the required capacity. The concrete floor slab must be capable of resisting the negative moments induced at the new support locations.

23.4.11. Deck Replacement

Precast concrete slabs are prefabricated as traditionally reinforced, prestressed, or a combination of the two. Panels are placed on the tops of the beams in a mortar bed with space or opening for shear connectors.

Cast-in-place (CIP) concrete slabs are cast on the tops of the beams encompassing the existing rehabilitated shear connectors or new replacement shear connectors.

Traffic control is a critical issue when considering a deck replacement.

TYPE OF PROJECT	BRIDGE RAIL ACTION
Preventive Maintenance (PM)	Replacement, retrofit, or repair of existing rail is not required, except when damaged beyond repair. When damaged beyond repair replace rail in accordance with Figure 16-10.* Consider in-kind repair to address obvious deficiencies. * Replacement may be deferred to a later project
3R with Major Bridge Rehabilitation or Widening (Widening on one or both sides)	Provide railing in accordance with Figure 16-10.
3R with Minor Bridge Rehabilitation (No widening and no work affecting the existing bridge rail)	Replacement, retrofit, or repair of existing rail is recommended, but not required, except if damaged beyond repair or does not meet height requirements, then replace in accordance with Figure 16-10. The height requirements are: 27" for low speed roadways (45 mph and less) 32" for high speed roadways (greater than 45 mph)* * 1 inch less tolerance allowed

Table 23-1
Bridge Rail Upgrade Procedures

23.5. Concrete Superstructures

Chapter 14 provides a detailed discussion on the design of concrete superstructures. Many of the Chapter 14 design and detailing practices also apply to the rehabilitation of an existing concrete bridge.

23.5.1. Remove/Replace Deteriorated Concrete

A clean, sound surface is required for any repair operation; therefore, all physically unsound concrete, including all delaminations, should be removed.

Verify that the concrete not removed is capable of resisting its weight, any superimposed dead load, live load (if the bridge will be repaired under traffic), formwork, equipment, and the plastic concrete without the need for supplemental temporary support. The formwork should resist the plastic concrete without slipping or bulging.

23.5.2. Crack Repair

Attempt to identify and remediate the mechanism causing the concrete to crack. Section 23.4.3 discusses crack repair for bridge decks. Use similar techniques for concrete superstructures.

23.5.3. Grouting

Because of the availability of epoxy injection, do not use grouting in crack repair unless the crack width is greater than 10 mm. Limit its application to filling post-tensioning ducts and to provide mortar-beds for precast concrete deck components, barriers, and bearings.

23.5.4. Post-Tensioning Tendons

The addition of post-tensioned tendons can restore the strength of prestressed concrete girders where original strands or tendons have been damaged. Strengthening by post-tensioning may also be applied to non-prestressed concrete girders.

Collision of over height vehicles or equipment with a bridge constructed with prestressed concrete girders may result in damage to or severing of the girder tendons. Exposure to water and salt may also cause damage, particularly where the concrete cover is damaged or cracked. Because the steel tendons determine the load-carrying capacity of the girder, any damage impairs resistance and must be repaired. The bridge engineer can use external longitudinal post-tensioning along the sides of pier caps to close transverse cracks and improve seismic performance.

At a minimum, the following steps apply:

- Conduct an investigation on the extent of damage.
- Perform a structural evaluation to determine the extent of repair.
- Evaluate the existing diaphragms to ensure their adequacy to support the end anchorage of the tendons.
- Determine the placement of the temporary load to be applied to the bridge prior to removal and placement of concrete in prestressed concrete girders, if any, to ensure the proper distribution of loads in the final condition.

Design the post-tensioning system in accordance with the manufacturer's recommendations. Wedge-type anchorages are susceptible to high seating losses for short-length tendons. High-strength prestressing bars are preferred in this application.

23.5.5. Bearings

Often, the existing bearings may only need cleaning or repositioning. Extensive deterioration or frozen bearings may indicate that the design should be modified. The bridge engineer may substitute a variety of elastomeric devices for sliding and roller bearing assemblies. If the reason for deterioration is a leak in the expansion joint seal, the seal should be repaired or replaced.

If the bearing is seriously dislocated, its anchor bolts badly bent or broken, or the concrete seat or pedestal is structurally cracked, the bridge may have a system-wide problem usually caused by temperature or settlement that warrants investigation.

See Chapter 19 for more information on bearings.

23.5.6. Design for Damaged Prestressed Concrete Beams

Definitions of High Load Damage[1]

Surface Damage: These are surface scrapes and small nicks less than one inch deep. This type of

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[1] Waheed, Kowal, Loo, Page2

damage does not generally warrant repairs unless it is associated with other bridge maintenance repairs.

Minor Damage: Isolated concrete cracks, nicks, and spalls up to one inch deep with no reinforcing or prestressing strands exposed. Minor damage adversely affects the aesthetics; however, the structural capacity is not reduced. It is important to restore concrete cover to prevent reinforcing steel from eventually becoming exposed and corroded.

Moderate Damage: Concrete cracks and wide spalls exposing reinforcing steel and prestressing strand. There is no immediate effect on the structural capacity, although cracks and exposed reinforcement can reduce structure life due to corrosion and freeze thaw action.

Severe Damage: This includes damaged prestressing strands and reinforcing steel, significant loss of cross section, and possible lateral misalignment due to girder distortion.

Cracks[2]

Use the following definitions for crack width:

- Hairline – less than 0.01 inch
- Narrow – less than 1/64th inch and greater than or equal to 0.01 inch
- Medium – less than 1/32nd inch and greater than or equal to 1/64th inch
- Wide – greater than or equal to 1/32nd inch

Repair/Replacement Criteria

Surface Damage. Surface damage does not, in general, need to be repaired. If work is being done on the bridge, surface damage repair can be included in the other work if desired.

Minor Damage. Repair minor damage generally by epoxy injection. Cracks greater than 0.010 inch in width should be pressure injected full depth with an approved epoxy resin. Prestressed girders should be in compression in their unloaded state; no cracks should be visible in the unloaded state, thus all cracks indicate some amount of damage.

Moderate Damage. Moderate damage may be repaired without preloading the girder or tensioning any strands; however, preload is recommended[3].

Severe Damage. Severe damage requires repair or replacement of the girder.

Repair Criteria

For a precast, prestressed girder to be repaired, it must meet the following criteria:

- Less than 25 percent of the total number of prestressed strands damaged unless exterior post-tensioning is planned.[4]
- No abrupt lateral offsets have occurred in the girder due to impact damage.[5]
- The shear capacity of the proposed repaired section is greater than the demand anticipated.

Prestressed Tendon Repair

The following repair procedure was developed for 0.5-inch strand. Should a damaged 0.6-inch strand require repair, confirm that the same methods, procedures, equipment, and supplies are applicable.[6]

Remove unsound concrete from around the damaged area, enough to expose sufficient undamaged portions of damaged strands to allow splicing operations as per manufacturer's recommendations. Note that many manufacturers recommend staggering splices if multiple strands are broken; thus, if the manufacturer recommends, it may be necessary to remove additional sound concrete to provide enough exposed strand to install a staggered splice. If a previously damaged strand is damaged again and the previous splices exposed by the new damage, cut out the old splice, cut the strand back far enough to splice to undamaged strand. If a new piece of strand must be spliced in, the couplers must be at least 3 feet apart as measured from the end of the couplers closest to each other.[7]

Install couplers/repair strands in such a way as to retension the strand to 60 percent of the specified ultimate strand strength.[8] For 0.5-inch, 270 ksi strand, this would be 162 ksi or 24.79 kips. See the manufacturer's instructions for methods to achieve this force.

[2] Waheed, Kowal, Loo, Page 5
[3] Shanafelt, Horn, Page A-52
[4] Shanafelt, Horn, Page 29
[5] Shanafelt, Horn, Page A-21 through A-22
[6] Banse, October 3, Harton, October 9
[7] Waheed, Kowal, Loo, Page 21
[8] Waheed, Kowal, Loo, Page 9

Stressing methods must be submitted to the bridge engineer for approval.

It is very important to check that all threaded components are free from defects and protect them from damage.

All threaded components should be lubricated prior to use.[9]

After stressing the strand, preload the girder, then unload the girder, and check the stress in the strand. If the strand is still tensioned as required, preload the girder and patch the concrete.

If the strand has lost more than 1 kip pretension, repeat the cycle of preload and retension until the loss is less than 1 kip. This is the slippage due to anchorage set. Once the anchorages have been set, place the girder and patch the concrete.[10]

Post-tensioned Tendon Repair

Use special care when repairing post-tensioned tendons. According to the main supplier the DOT&PF has historically used, post-tensioned tendons cannot be repaired without special equipment. Follow the manufacturer's recommendations for post-tensioned tendon repair.[11]

See Appendix 23A for the DOT&PF procedure for the construction repair of damaged prestressed concrete beams and a list of the cited references.

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[9] Zobel, Jirsa, Page 82, Harton, October 9

[10] Zobel, Jirsa, Page 81

[11] Harton, October 9

23.6. Steel Superstructures

Chapter 15 provides a detailed discussion on the structural design of steel superstructures for new bridges. Many of the Chapter 15 design and detailing practices also apply to the rehabilitation of an existing steel superstructure.

23.6.1. Fatigue-Crack Retrofit

Fatigue damage entails the formation of cracks in base metal or welds. If not repaired in a timely manner, fatigue cracks can lead to brittle fractures. The type of repair and its timing depend on many factors including:

- reason for the cracking (e.g., poor detailing, heavier than anticipated truck traffic, poor notch-toughness, load induced or distortion induced, constraint);
- location of the crack (e.g., cross frame, stiffener, weld, heat-affected zone, main member);
- depth, length, and geometry of the crack; and
- redundancy.

23.6.2. Fatigue-Cracking Countermeasures

Hammer Peening

Peening is an inelastic reshaping of the steel at the surface location of cracks, or of potential cracks, by using a mechanical hammer. This procedure not only smoothes and shapes the transition between weld and parent metal, it also introduces compressive residual stresses that inhibit the cracking. Peening is most commonly used at the ends of cover plates to reduce fatigue potential. However, the success of hammer peening is highly dependent upon the skill of the operator.

Ultrasonic Impact Treatment (UIT)

A computer-controlled peening process using high-speed peening called ultrasonic peening is available. This removes the dependency of the quality of mechanical-hammer peening on the operator's proficiency. This process promises weld-toe enhancement for unavoidable poor fatigue resistance details such as terminations of longitudinal stiffeners. It involves the deformation of the weld toe by a mechanical hammering at a frequency of around 200 Hz superimposed by ultrasonic treatment at a frequency of 27 kHz. The objective of the treatment is to introduce beneficial compressive residual stresses at the weld toe by plastic deformation of the surface and to reduce stress concentration by smoothing the weld toe profile.

For more information, see "Fatigue Resistance of Welded Details Enhanced by Ultrasonic Impact Treatment (UIT)" by Sougata Roy, John W Fisher, Ben T Yen in the International Journal of Fatigue, Volume 25, Issues 9–11, September–November 2003, Pages 1239–1247.

23.6.3. Section Losses

The following options are available to correct section losses by adding doubler plates:

1. **Welding Doubler Plates**. It is common practice to use welding for shop fabrication of steel members and for welding pieces in preparation for rehabilitation work. Field welding is often difficult to perform properly in high-stress areas, and this requires individuals with the necessary skill and physical ability. The proper inspection of field welds is equally difficult. Preferably, use a shop weld instead of a field weld.

   Field welding should only be allowed on secondary members, for temporary repairs, or in areas where analysis shows minimal fatigue stress potentials.

2. **Bolting Doubler Plates**. Bolting doubler plates over a corroded section (that has been cleaned and painted to prevent further section loss) is a more reliable long-term solution.

23.6.4. Add Cover Plates

If the deck is deteriorated and removed, adding cover plates to strengthen a beam may be a viable strategy. However, the *LRFD Specifications* place the ends of fully welded cover plates into Fatigue Category E or E'. The advantages of adding cover plates may be offset by introducing fatigue-prone details. If bolts designed in accordance with LRFD Article 6.10.12.2.3 are used at the end of the cover plates, apply Fatigue Category B. Because this requires the presence of drilling equipment and work platforms, consider a fully bolted cover plate construction.

23.6.5. Introduce Composite Action

Introducing composite action between the deck and the supporting beams is a cost effective way to increase the strength of the superstructure. The *LRFD Specifications* mandate the use of composite action where current technology permits. Welded studs or high-strength bolts can achieve composite action. Design shear connectors in accordance with LRFD Article 6.10.10.

Composite action considerably improves the strength of the upper flange in positive moment areas, but its beneficial effect on the beam as a whole is only marginal. The combination of composite action in conjunction with selective cover plating of the lower flange is the most effective way of beam strengthening.

Introducing composite action near joints prevents the deck from separating from the beams, thus increasing the service life of the deck. Provide this on each bridge that will have its deck removed for other reasons.

23.6.6. Add New Girders

If the deck is removed, a new set of girders added to the existing bridge is one alternative to strengthen the superstructure. To ensure proper distribution of live load, rigidity of the new girders should be close to that of the existing ones.

23.6.7. Bearings

The discussion in Section 23.5.5 also applies to steel superstructures.

23.6.8. Painting

Technically, bridge painting is maintenance work and not rehabilitation work, but frequent painting is considered in conjunction with rehabilitation work on steel structures.

Painting is generally warranted for bridges experiencing severe corrosion with section loss. It should also be considered for highly visible bridges with rust Grade 4 or more (SSPC Vis 2) where 10 percent of the surface area is rusted. This level of corrosion is aesthetically unacceptable and will progress to steel section loss.

Much of the interior of Alaska has a benign climate with slow corrosion rates where zone (spot repair) painting at expansion joints may be appropriate. Complete superstructure re-painting may be more appropriate for bridges located in coastal areas having a more aggressive climate with accelerated corrosion rates. Consider the following three options:

1. full removal of existing paint and repainting,
2. a complete recoat over the top of the existing paint (overcoat), or
3. zone (spot repair) painting.

An important factor is that virtually all paint applied to bridges prior to 1988 contains lead and other heavy metals. To remove existing paint, the current state of practice is abrasive blast removal, full enclosure, with environmental and worker monitoring.

The paint industry has developed products that can be successfully applied over existing paints. An overcoat may be an economic alternative to full removal and repainting where a uniform appearance is desired at the conclusion of the project; however, the removal of the lead-based paint is deferred until a subsequent rehabilitation or structure replacement. Zone painting neither provides a uniform appearance nor removes the lead-based paint. Zone painting may be appropriate in localized areas where corrosion could cause section loss.

Consider the proper selection of paint for an overcoat. An improperly specified or improperly applied overcoat can cause failure of the original paint that was performing satisfactorily. Review the manufacturer's literature on any paint's service environment and recommended application environment.

Overcoating lead-based paint having extensive rust spots but less than 1 percent of surface rusted (rust Grade 6) is generally not cost effective. The existing top coat is usually aluminum-based and too stiff and brittle to overcoat.

Blast cleaning must remove rust spots to bare metal and remove the aluminum top coat. Full containment is required.

Overcoating retains the existing lead-based paint and removal is merely deferred. It is generally more practical to postpone painting until the bridge has reached rust Grade 4 (10 percent of surface rusted) when complete lead-based paint removal and repainting is appropriate.

Overcoating existing zinc-based paint for rust Grade 6 will extend the service life of the existing coating. This option is especially attractive on bridges that are highly visible to the public.

DOT&PF typically specifies a single-component, moisture-cure, polyurethane paint system meeting applicable ASTM specifications. The specific paint must allow application in cold and damp conditions.

23.6.9. Heat Straightening

This technique is restricted to hot-rolled steels. Steels deriving their strength from cold drawing or rolling tend to weaken when heated. The basic idea of heat straightening is that the steel, when heated to an appropriate temperature, loses some of its elasticity and deforms plastically. This process rids the steel of built-up stresses.

Heat straightening is as much an art as science. To avoid overheating the steel, this technique should only be performed by those with experience in the process.

Heat straightening temporarily reduces the resistance of the structure. Apply measures such as vehicular restriction, temporary support, temporary post-tensioning, etc., as appropriate while the work is in progress.

For additional guidance on heat straightening, see *Guide for Heat-Straightening of Damaged Steel Bridge Members*, FHWA, 2008, and *Heat-Straightening Repair of Damaged Steel Bridge Girders: Fatigue and Fracture Performance*, NCHRP, 2008.

23.6.10. Pin/Hanger Rehabilitation

Bridge engineers originally used pin and hanger details to facilitate the analysis of bridges by providing pins in otherwise continuous bridges. Today their use is not necessary due to modern computer-based structural analysis.

These details are particularly susceptible to corrosion. Corrosion can result in the initiation of fatigue cracking in the hangers due to frozen pins and the unseating of the hangers on the pins due to misalignment from the corrosion product.

The infamous collapse of one span of the Mianus River Bridge on I-95 in Connecticut was the result of corrosion of a pin and hanger detail.

Three solutions are possible for pin and hanger details:

1. **Unlock Frozen Pins and Hangers.** The pin and hanger detail can be disassembled after providing alternative support to the suspended girder. Then, the various components of the detail can be cleaned of rust and dirt or replaced before re-assembly.

2. **Provide a Catch Girder.** As a safeguard against failure, especially for fracture-critical girders, an alternative permanent support system can be fabricated to "catch" the suspended girder ends if the pin and hanger detail fails. Such a structure must be temporarily provided to perform the unlocking of frozen details discussed above.

3. **Eliminate the Pin and Hanger Detail.** If the girder sections allow, a bolted splice of the web and flanges can be fabricated to replace the pin and hanger. A structural analysis of the resulting continuous structure must verify that the resulting loads do not exceed the resistance of the existing girder section.

23.6.11. Post-Tensioning

External post-tensioning can be applied to both steel and concrete beams to reduce tensile stresses, to strengthen beams, or to make simply supported beams continuous. There are a variety of successful methods of post-tensioning in the literature.

The *LRFD Specifications* require the establishment of resistance at ultimate limit states at which the interaction between the parent and the post-tensioning systems should be investigated.

Because they are always close to the beam ends, post-tensioning anchorages are vulnerable to salt-laden water seeping through imperfectly sealed deck joints. Protect the tendons by corrosion-resistant ducts, either grease filled or grouted, especially if being exposed to airborne salts such as at overpasses.

23.7. Substructures/Foundations

Chapter 18 provides a detailed discussion on the structural design of substructures for new bridges, and Chapter 17 applies to foundations. Many of the Chapter 17 and Chapter 18 design and detailing practices also apply to the rehabilitation of the substructures of an existing bridge.

23.7.1. Deadman Anchorages

The lateral force exerted by retained earth tends to push forward and rotate abutments and retaining walls. One solution for this problem is the installation of a deadman anchor.

A deadman is a heavy solid mass, usually concrete blocks that are connected to the retaining structure by long steel rods. A deadman is located in a stable earth mass well behind the structure. For wingwalls, or walls located on both sides of the roadway, they can simply be connected together by steel tension rods.

Protect the rods against corrosion, and consider the effects of differential settlement.

Because this stabilization technique modifies the wall support from a cantilever to simple span pinned, check the wall reinforcement for the revised moments. The lateral earth pressure diagram may also be changed if more than one level of tension rods and anchors are installed.

23.7.2. Column Repair/Rehabilitation

The typical repair of concrete columns is to place a concrete jacket around the member to protect it from further deterioration or to restore its structural integrity.

The repair can be made with a standard wood or metal form work which is removed after curing or a fiberglass form that remains in place and helps protect the surface of the member.

In areas of moderate to high seismicity, consider installing a grouted steel jacket to both repair the column and provide enhanced seismic performance.

23.7.3. Post-Tensioning

Inadequately reinforced concrete pier caps may require strengthening by external post-tensioning.

Evaluate the existing concrete in the cap. Include tensioning strand or rods externally on the cap to add compression to the cap. Use brackets, distribution plates, and other components to transfer the post-tensioning forces to the cap. If aesthetics are a concern, widen the cap with ducts placed internally for the post-tensioning.

Post-tensioning is usually symmetrical to the cap so that an eccentric force is not introduced. Look at the stressing sequence to ensure that the cap is not overloaded eccentrically during post-tensioning operations.

23.7.4. Pile Section Loss Repair/Replacement

For steel piles, the following applies to section losses:

1. **Small Loss.** The restoration of the section of piles that experience a small loss of section associated with "normal" rusting is usually not warranted.

2. **Medium Loss.** When rusting has reduced the section of the pile such that it becomes a structural concern, the missing cross section is rebuilt by adding plates to the flanges and/or web as appropriate by either welding or bolting. Concrete cast in a stay-in-place fiberglass form can also be considered.

3. **Extensive Loss.** When the pile has deteriorated such that there is insufficient sound remaining material for the section to be rebuilt, use a new pile; the damaged pile may or may not be removed.

For wood piles, repair section losses by partial replacement, epoxy injection, jacketing, or some combination of these methods.

For information on wood piles, see "Timber Bridges – Design, Construction, Inspection and Maintenance" by M. A. Ritter, United States Department of Agriculture, Forest Service, EM 7700-8, June 1990, Chapter 14.

For concrete piers, section loss may be repaired by removing all deteriorated material, constructing a formwork for a jacket, placing a reinforcing steel cage of appropriate size in the formwork, and filling it with compacted concrete. The technique has extensive literature on its application.

23.7.5. Crutch Bent Repair

Rehabilitate settled or otherwise failing piers or abutments through a crutch bent. A crutch bent is a supplemental bent placed adjacent to a failing pier or abutment to restore support.

23.7.6. Tremie Concrete Encasement

When underwater footings or pile caps are undermined, one of the more common repair methods is to fill the void foundation area with a concrete. To place the concrete, use some type of formwork to confine it. A tremie encasement is a steel, wood, or concrete form that is placed around the existing footing or cap to reestablish the foundation. The form allows the concrete to be pumped under the eroded footing and displaces the water in the encasement through vents.

23.7.7. Concrete Bridge Seat Repair or Extension

Concrete bridge seats can fail due to deterioration of concrete, corrosion of the reinforcing bars, friction from the beam or bearing devices sliding directly on the seat, and the improper design of the seat which results in shear failure. Anchoring an extension to the existing cap restores adequate bearing for beams that have deteriorated or sheared at the bearing. The extension should not be exposed to any load during curing. Also, repair any damage to the end of the beam itself.

23.7.8. Wingwall Repair

In many old concrete abutments, the wingwalls tend to break-off and to separate from the main body due to earth pressure and differential settlement. If the opening has been stable, the do-nothing option may be acceptable. If not stable, remove the wingwalls and completely rebuild. Footings for the new wingwalls should be at the same level as that of the main body.

23.7.9. Micropile Underpinning

Micropiles, also known as minipiles and pin piles, are small-diameter reinforced piles that are drilled and grouted to support structures. These piles may reach service loads up to 300 tons, can be installed to depths of approximately 200 feet, and usually use some type of steel bar or bars and/or steel casing pipe. The bars are grouted into the ground and/or the casing pipe is filled with grout.

Although a conventional pile is generally quite large and requires heavy equipment and large staging areas for installation, micropiles can be used where conventional piling is not convenient or possible, such as for underpinning or retrofitting existing bridges or structures.

Micropiles have proven effective in many ground improvement applications by increasing the bearing capacity and reducing settlement, especially when strengthening the existing foundations.

23.8. Seismic Retrofit

23.8.1. Seismic Evaluation

The ability to predict the forces developed by an earthquake is limited by the complexity of predicting the ground acceleration and displacement, and the associated response of the structure. The motion can generally be described as independent rotation, in any direction, of each bridge abutment or pier, in or out of phase with each other, combined with sudden vertical displacements. The ground between piers can distort elastically and in some cases rupture or liquefy.

Historically, bridge failures induced by the motions of the abutments and piers stem from two major inadequacies of many existing bridge designs — the lack of adequate connections between segments of a bridge and inadequately reinforced columns. Other deficiencies include inadequately reinforced footing and bent cap concrete and inadequate design force levels considering the likelihood of earthquakes at the location.

Fortunately, tying the segments of an existing bridge together is an effective means of preventing the most prevalent failure mode — spans falling off the supports. It is also the least expensive of the seismic deficiencies to correct. Bridges with single-column piers are particularly vulnerable where segments are not connected.

Column failures have historically been associated with inadequate confinement, shear, and anchorage reinforcement. Confinement and shear deficiencies have occurred where too few and/or improperly detailed ties and spirals are present. Short-lapped splices in longitudinal column reinforcing have resulted in anchorage failures. These failure modes are particularly critical in single-column piers.

Determining the preferred retrofit strategy involves the following considerations:

- anticipated failure mode
- influence on other parts of the bridge under seismic and normal loadings
- interference with traffic
- fabrication and installation cost

Some retrofit strategies are designed to correct bridge inadequacies related to seismic resistance. The procedures may be categorized by the function the retrofit serves, including:

- restraining uplift,
- restraining longitudinal motion,
- restraining hinges,
- widening bearing seats,
- isolating/modifying seismic forces between the superstructures and substructures (seismic isolation bearings),
- strengthening columns and footings, and
- restraining transverse motion.

For seismic retrofits as part of 3R projects, determine the Seismic Performance Zone in accordance with the *LRFD Specifications.* Bridges in Seismic Zone 1 are not required to be retrofitted. Retrofit bridges in Seismic Zones 2, 3, and 4 as needed based on this *Manual* and the FHWA *Seismic Retrofitting Manual for Highway Bridges*, May 1995. Essential bridges will be identified by the Chief Bridge Engineer.

23.8.2. Typical DOT&PF Practices

Highway bridges are vulnerable to partial or total collapse during earthquakes due to three main reasons:

- girders dropping from their supports
- seismic moments and shears exceeding the capacity of the columns
- ground failure associated with liquefaction and lateral spread

Seismic retrofit attempts to improve the performance of existing bridges that are vulnerable to these demands. Retrofits typically fall into one of two categories: Phase I or Phase II.

Phase I Seismic Retrofit

The objective of Phase I retrofit strategies is to prevent girders from falling off their supports (abutment and pier seats) in a relatively cost-effective manner.

Tie the girders to each other and to their supports with restrainer cables to limit their displacement relative to the supports.

Seismic retrofit can also limit girder movement by installing concrete shear keys. On some bridges, installing timber blocking between the ends of the girders and the abutment backwall limits longitudinal girder movement and attracts seismic forces away from the piers to the abutments.

Another strategy involves increasing the support width, thus increasing the displacement capacity of the system.

Phase II Seismic Retrofit

Categorize all work beyond Phase I as Phase II retrofit strategies. These Phase II strategies are generally much more expensive. They typically address column seismic deficiencies through retrofits such as column jacketing or foundation deficiencies through retrofits such as footing modifications.

Seismic isolation is generally categorized as a Phase II retrofit.

23.8.3. Retrofit Techniques

Column Jacketing

Jacketing consists of adding confinement to columns by covering with a grout-filled steel shell, fiberglass wrap, or carbon fiber wrap.

The steel jacket consists of structural steel welded over the column and grouted. The fiberglass and carbon fiber wraps are glued to the column in multiple layers. These wraps are proprietary products.

Non-circular columns can be retrofitted by jacketing, but the increased rigidity must be evaluated. A circular steel casing may be placed around the non-circular column and grouted.

Locate jacketing only at the points of potential column hinge formations. However, if more than half the total height of the column requires a jacket, typically extend the jacket to full height for improved aesthetics.

Jacketing increases column rigidity, amplifying global seismic forces and attracting more load to the column. The bridge engineer typically needs to evaluate this increased rigidity.

Seat Width Enlargement

Seat width extensions allow larger relative displacements to occur between the superstructure and substructure before support is lost, and the span collapses.

The seat width extension strategy at piers is typically selected when the pier has inadequate strength to resist the forces from restrainer cables connecting the superstructure to the pier. The extensions are likely to be exposed to large impact forces due to the dropping span; therefore, they should either be directly supported by a footing or be adequately anchored to the abutment or pier cap.

Follow the provisions in the *LRFD Specifications* relative to the design of seat widths, as practical.

DOT&PF typically uses a combination of post tensioning rods and epoxy-bonded dowels for pier cap widenings.

Post tensioning rods are generally impractical at abutments due to the need to excavate behind the backwall and the resulting disruption of traffic. Seat width dimensions are frequently controlled by the development length of the epoxy-bonded dowels. Increase the vertical load by 100 percent (200 percent of dead load) to account for impact effects.

Structural Continuity

Some older bridges were constructed with multiple simple spans. They lack longitudinal continuity. These older bridges frequently have minimal support lengths under the ends of the girders and limited restraining devices (e.g., anchor bolts).

Historically, limited support lengths, inadequate restraining devices, and lack of superstructure continuity have contributed to spans collapsing during earthquakes.

It is typically not feasible to retrofit a superstructure to make it continuous for full dead and live loads unless the deck is being replaced; however, steel plate girder bridges can sometimes have web-continuity plates added over the piers. These plates and their connections to the existing girder ends should be designed for increased vertical loads (200 percent of dead load reaction) to account for impact effects.

Restrainers and Ties

In general, restrainers are add-on structural devices that do not participate in resisting other than seismic force effects. Typically, these components are made of steel, design them to remain elastic during seismic action, and exercise special care to protect them against corrosion.

There are three types of restrainers — longitudinal, transverse, and vertical. The purpose of the two former types is to prevent unseating the superstructure. The objective of the third type is to preclude secondary dynamic (impact) forces that may result from the vertical separation of the superstructure. The restraint devices should be compatible with the geometry, strength, and detailing of the existing structure.

Ties are restrainers that connect only components of the superstructure together. They are activated only by seismic excitation.

Shear keys or blocks allow in-service movements of the bridge, without applying significant loads to the substructure. During an earthquake, the stoppers transmit the seismic force to the substructure. Transverse stoppers are used frequently in seismically isolated bridges. Longitudinal stoppers are uncommon due to the larger in-service longitudinal movements of bridges.

Steel Rocker Bearings

For bridges within Seismic Zones 2, 3, and 4 (SDC B, C, and D), the retrofitting measures include modification or elimination of existing steel rocker bearings. Major reconstruction projects in Zone 1 (SDC A) may also be good candidates for the elimination of existing steel rocker bearings, which will be decided on a case-by-case basis.

Rocker bearings are replaced with steel reinforced elastomeric bearing pads or seismic isolation bearings. Steel rocker bearings are typically much taller than the replacement devices (e.g., pads, isolators).

This height difference necessitates a reinforced concrete fill be cast between the top of the existing cap beam and the underside of the new bearing devices.

Seismic Isolation Bearings

There is a broad variety of patented seismic isolation bearings that are commercially available. They permit either rotation or translation or both. They have special characteristics by which the dynamic response of the bridge is altered, and some of the seismic energy is dissipated. The primary change in structural response is a substantial increase in the period of the structure's fundamental mode of vibration. The devices are designed to perform elastically in response to normal service conditions and loads.

DOT&PF typically uses only friction pendulum bearings because they are relatively unaffected by temperature changes. Sole source procurement policies need to be followed when specifying these proprietary devices.

See the AASHTO *Guide Specifications for Seismic Isolation Design* and the FHWA *Seismic Retrofitting Manual for Highway Structures*, FHWA, 2006 for more information.

Modifying Seismic Response

Use the following techniques to modify the seismic response of a bridge:

1. **Flexural Reinforcement.** Because of conservative provisions, concrete columns have often been both over-designed and over-reinforced in the past.

   Over-reinforcement means that the flexural steel is not expected to yield during the design event, resulting in both higher compressive and shear forces on the concrete. If other design criteria permit, some of the flexural steel may be cut to induce yield. If circumstances warrant, the flexural reinforcement may be increased.

   Locate the vertical bars in a concrete jacket that is shear connected to the column by means of drilled and grouted dowels. This also increases the rigidity of the column, potentially rendering it counterproductive.

2. **Infill Shear Wall.** A concrete shear wall can be added between the individual columns of pier.

   If the existing footing is not continuous, it should be made so.

   Connect the wall to the columns by means of drilled and bonded dowels. This method substantially changes the seismic-response characteristics of the structure, requiring a complete reanalysis.

   The more rigid infill wall may attract more load, and this increase must be considered in the design.

23.9. Bridge Widening

23.9.1. General Approach

A bridge widening can present a multitude of challenges during the planning and design stages, during construction, and throughout its service life. Pay special attention to both the overall design and detailing of the widening to minimize construction and maintenance problems.

This section presents DOT&PF guidelines for widening existing bridges. The following briefly summarizes the basic objectives in bridge widening:

- Match the structural components of the existing structure, including splice locations.
- Match the existing bearing types in terms of fixity.
- Do not perpetuate fatigue-prone details.
- Evaluate the need to replace the bearings and joints in the existing structure.
- Evaluate the load-carrying capacity of the existing structure.
- Evaluate the seismic resistance of the existing and widened structure. Incorporate retrofit measures if appropriate.
- Use the same structure frame on the widened portion as on the existing bridge.
- Match the flexibility/stiffness of the existing and new superstructures.

23.9.2. AASHTO Standards

It is not normally warranted to modify the existing structure solely because it was designed to AASHTO Specifications prior to the adoption of the *LRFD Bridge Design Specifications* and its latest interim changes.

When preparing plans to modify existing structures, it is often necessary to know the live load and stress criteria used in the original design. With few exceptions, structures on the Alaska highway system have been designed for loads and stresses specified by AASHTO.

The bridge engineer should be aware of the historical perspective of design criteria, such as live loads, allowable stresses, etc., when analyzing a rehabilitated structure. For accurate and complete information on specific structures, see the General Notes on as-built plans, old standard drawings and special provisions, and the appropriate editions of the AASHTO Specifications.

23.9.3. Details of Existing Structures

Load-Carrying Capacity

An existing structure may have been originally designed for either live loads or seismic loads less than those currently used for new bridges.

If such a structure becomes a candidate for widening, consult the data available in the Alaska Bridge Inventory on the condition of the existing structure. Determine a load rating for the existing bridge to quantify the capacity of the existing bridge (see Chapter 27). Based on this information, the bridge engineer will determine whether the existing structure should be strengthened to increase load-carrying capacity. For this evaluation, consider the following:

- cost of strengthening existing structure
- physical condition, operating characteristics, and remaining service life of the structure
- seismic resistance of structure
- other site-specific conditions
- only structure on route that restricts permit loading
- width of widening
- traffic accommodation during construction

Rolled Steel Beams

Throughout the years, modifications to rolled steel beam sections have occurred. Bridge engineers should refer to the construction-year AISC steel tables for rolled beam properties and other data.

Materials

For material properties of older structures, check the General Notes on the existing bridge plans, if they exist, or plans of comparable bridges of the day. Also, reference the MBE for historical properties of materials.

Sometimes, the grade of reinforcing steel is indicated as “intermediate grade”; this terminology means Grade 40.

Until approximately 1960, ASTM A7 was the primary structural steel used in bridge construction. The yield and tensile strengths of A7 may be taken as 33 ksi and 66 ksi, respectively.

23.9.4. Girder Type Selection

In selecting the girder type for a structure widening, the widened portion of the structure should be a construction type and material type consistent with the existing structure. Proportion the widening to ensure that the structural response is similar to the existing bridge.

23.9.5. Deck Closure Pour

Where dead load deflection exceeds ¼ inch, allow the widening to deflect and use a closure pour to complete the attachment to the existing structure.

A closure pour serves two useful purposes: It defers final connection to the existing structure until after the deflection from the deck slab weight has occurred; it provides the width needed to make a smooth transition between differences in final grades that result from design or construction imperfections.

The bridge plans should include a note indicating the required waiting period, if any, between deck concrete and closure concrete placement.

23.9.6. Vehicular Vibration During Construction

Structures deflect when subjected to live loading, and many bridge widenings are constructed with traffic on the existing structure. Fresh concrete in the deck is subjected to deflections and vibrations caused by traffic. Studies such as NCHRP 86 *Effects of Traffic-Induced Vibrations on Bridge-Deck Repairs* have shown that:

- Good-quality reinforced concrete is not adversely affected by jarring and vibrations of low frequency and amplitude during the period of setting and early strength development.
- Traffic-induced vibrations do not cause relative movement between fresh concrete and embedded reinforcement.
- Investigations of the condition of widened bridges have shown satisfactory performance of attached widenings, with and without the use of a closure pour placed under traffic.

23.9.7. Substructures/Foundations

Investigate foundation capacities of existing structures if additional loads will be imposed on them by the widening. Settlement of newly constructed footings under a widened portion of a structure is possible. The new substructure could be tied to the existing substructure to reduce the potential for differential foundation settlements, provided that this does not adversely affect the existing substructure. If the new substructure is not tied to the existing substructure, make suitable provisions to prevent possible damage where such movements are anticipated.

Work with Statewide Materials to assess the compatibility of new and existing foundations and the potential for differential settlement.

23.10. Post-Earthquake Repairs

23.10.1. Post-Earthquake Column Repair

Prepare repairs to columns damaged by earthquakes in accordance with the methodology outlined in the "Repair of Reinforced Concrete Bridge Columns via Plastic Hinge Relocation, Volume 3: Design Guide" by Krish et al (2018). Additional column repair details may be implemented in accordance with "Rapid Seismic Repair of Column to Footing Connections – Phase 2" by Brodbeck et al (2002).

Appendix 23.A

REPAIR OF DAMAGED IN-SERVICE PRESTRESSED CONCRETE BEAMS

23.A.1 Materials[1]

23.A.1.1 Epoxy for Crack Injection

Meet the requirements of ASTM C881 Type IV, Class B or C for epoxy for crack injection.

23.A.1.2 Concrete Bonding Agent

Meet the requirements of ASTM C1059 Type II for bonding agent used for concrete spall repairs.

23.A.1.3 Reinforcing Steel

Conform to *Alaska Standard Specifications*, Section 503 for reinforcing steel. Use ASTM A706 Grade 60 reinforcing steel. Reinforcing steel coating should match that which was originally in the area being repaired. Repair all damaged coatings on salvaged bars.

23.A.1.4 Concrete[2]

For the concrete repair material use a preblended, prepackaged cement-based mortar requiring only the addition of potable water. Do not use material that contains any chlorides or lime other than amounts contained within the hydraulic composition. Use a concrete repair material with sufficient compressive strength as defined by ASTM C109 to reach the original girder's f'_{ci} within the time the preload is applied to the girder. Achieve the bond strength of the concrete repair material when tested according to ASTM C882 that meets the requirements in the table below.

Original Beam 28-day Compressive Strength (f'_c) (psi)	Required Bond Strength (psi)
4000	285
5000	320
6000	350
7000	380
7500	390
8000	405
8500	415

23.A.1.5 Couplers and Tensioning Devices

All couplings must develop at least 95% of the specified minimum ultimate strength of the prestressing steel without permanent deformation. Localized yielding of coupling components is permitted; however, generalized permanent yielding is not permitted. The coupling of tendons must not reduce the elongation at rupture below the requirements of the tendon itself.

23.A.2 Repair Procedures

23.A.2.1 Patching Concrete

[1] Waheed, Kowal, Loo, Page 8

[2] Based on currently used patch material as reported by Banse, October 3

Clean all patch joints of surface laitance, curing compound and other foreign materials before fresh concrete is placed against the surface of the joint. Roughen the existing concrete surface to a full amplitude of approximately 1/4 inch by abrasive blasting or mechanical equipment. Do not discharge pulverized concrete and waste sand from abrasive blasting into streams. Remove the pulverized concrete and waste sand from abrasive blasting from work platforms and dispose of properly. Make saw cuts three quarters of an inch deep in adjacent sound concrete to provide a minimum depth of patch.[3] If the concrete is damaged by heat and/or fire, then take cores as directed by the engineer to determine the extent of the concrete damage. Use abrasive blast methods to clean all construction joints to the extent that clean aggregate is exposed. Flush all construction joints with water and allow to dry to a surface dry condition immediately prior to placing concrete. Remove and repair damaged stressing strand. Repair all damaged reinforcing steel by straightening or replacing as needed. Mechanical splice devices may be used as long as concrete cover is not compromised. Welding may not be done unless the original and the new bars are weldable, and welding procedures and certified welders are approved by the engineer. Repair damaged coatings on reinforcing steel according to *Alaska Standard Specifications* Section 503. Apply a concrete bonding agent prior to placing the patch.

Preload the girder before placing concrete patch[4]. Preload should be of a magnitude and location to produce no stress in the bottom face of the repaired girder. Preload should remain on the girder until the patched concrete attains sufficient strength to withstand the stresses of an unloaded girder. This load is generally very close to f_{ci}, the required strength at stress transfer of the original girder. In all cases, use f_{ci}; however, if unloading the girder at a lower strength is desired, calculations can be made to determine the required strength of the patch when unloaded.[5] Test the strength of the patched concrete material in accordance with AASHTO T 22 and T 23. Field cure the test specimens in accordance with AASHTO T 23, Section 10.2.

23.A.2.2 Crack Repair[6]

Cracks less than 0.013 inches in width need not be repaired. Preload should be applied prior to crack injection[7]. Repair shrinkage cracks on the perimeter of the patches in the same manner as other cracks.

Submit a detailed installation plan to the engineer for review and approval. Include at least the following information in the installation plan:

- Manufacturer's written recommendations for product use, surface preparation, temperature requirements, and all other product requirements relevant to use of the system in this particular application
- Schedule
- Temperature control method
- Surface and crack preparation procedure
- Method of applying the seal to concrete surface
- Mixing method
- Injection method
- Curing method and requirements

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[3] Shanafelt, Horn, Page A-53, Waheed, Kowal, Loo, Page 9

[4] Shanafelt, Horn, Page A-24, Waheed, Kowal, Loo, Page 11

[5] The required strength will be less than f_{ci} because the girder will have additional dead load than the weight of the girder used to determine f_{ci}. Additional dead loads include but are not limited to wearing surface, curb, rail, and diaphragms.

[6] *Alaska Standard Specifications for Highway Construction* Section 501-3.17

[7] Shanafelt, Horn, Page A-17

- Quality control program including Quality Control (QC) Inspector's name, qualifications, and manufacturer's written endorsement of the QC Inspector

Provide a manufacturer approved QC Inspector at the site from the time concrete surface preparation begins until the last crack has been injected with epoxy. Provide a written log of the crack repair procedure, prepared by the QC Inspector, to the engineer within 24 hours of completing the concrete crack repairs. All discrepancies between the installation plan and the QC Inspector's report must be corrected within seven days of completing the concrete crack repairs.

23.A.2.2.1 Preparation

Clean all cracks to be filled, so they are free of dust, silt, oil, and all other material that would impair epoxy bonding to the concrete. Use compressed air jets unless the jets will not remove material from within the cracks, in which case, flush the cracks with water under pressure. When flushing water is used, blow all water out of the cracks with oil-free compressed air before injecting epoxy. Do not use metal brushes, acids, or corrosives to clean concrete.

Insert suitable injection ports in the cracks at intervals equal to the thickness (±5%) of the concrete being injected. At the end of a crack, the first port shall be about half this distance from the end. The spacing of the ports shall be adjusted so that the epoxy substantially fills the cracks.

Seal the surface of the crack between ports with tape or other temporary surface sealant which is capable of retaining the epoxy adhesive in the crack during pressure injection and until the epoxy has hardened.

23.A.2.2.2 Injection

Pump the epoxy adhesive into the cracks through the injection ports. The pump, hose, injection gun, and appurtenances shall properly proportion and mix the epoxy and shall be capable of injecting the epoxy at a sufficient rate and pressure to completely fill all designated cracks. Use a suitable gasket on the head of the injection gun to prevent the adhesive from running down the face of the concrete. Keep pumping pressure as low as practicable.

The temperature of the concrete shall be not less than 40°F or greater than 80°F at the time epoxy is injected.

Before starting injection work and at hourly intervals during injection work when requested by the engineer, a three ounce sample of mixed epoxy shall be taken from the injection gun. Should these samples show any evidence of improper proportioning or mixing, injection work shall be suspended until the equipment or procedures are corrected.

The epoxy adhesive shall be forced into the first port at one end of a crack until adhesive runs in substantial quantity from the next adjacent port. The first port shall then be sealed and injection started at the next port. Injection shall then continue from port to port in this manner until the crack is fully injected. For slanting or vertical cracks, pumping shall start at the lower end of the crack. Where approximately vertical and horizontal cracks intersect, the vertical crack below the intersection shall be injected first. Seal the ports by removing the fitting, filling the void with epoxy and covering the area with tape or surface sealant.

Keep the sealing tape and temporary surface sealant in place until the epoxy has hardened.

23.A.2.2.3 Surface Finish

Remove all sealant tape and other temporary surface sealant when no longer required. Remove all spillage of epoxy.

Finish all exposed concrete surfaces to hide all evidence of the crack and epoxy injection system. Match the color, texture and appearance of the undamaged concrete surrounding the crack to the satisfaction of the engineer.

23.A.3 References

1. Waheed, Abdul; Kowal, Ed; Loo, Tom; *Repair Manual for Concrete Bridge Elements, Version 2.0*, Alberta Infrastructure and Transportation, Technical Standards Branch, Bridge Engineering Section, October 11, 2005, Government of the Province of Alberta.

2. Banse, Steve; *Re: Prestressed Girder Repair Procedures*, e-mail, Sent 2006, October 3, 10:08AKDT.

3. Shanafelt, G. O.; Horn, W. B.; *Evaluation of Damage and Methods of Repair for Prestressed Concrete Bridge Members, Preliminary Draft, Final Report* National Cooperative Highway Research Program Transportation Research Board, National Research Council, Project 12-21, December 1984.

4. Harton, Bruce, *Phone Conversation*, Prestressed Supply, Inc., Manufacturer's Representative, October 10, 2006.

5. AASHTO, *AASHTO Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, American Association of State Highway and Transportation Officials.

6. AASHTO LRFR, *AASHTO Manual for Condition Evaluation and Load Resistance Factor Rating (LRFR) of Highway Bridges*, American Association of State Highway and Transportation Officials.

7. AASHTO, *AASHTO LRFD Bridge Design Specifications*, American Association of State Highway and Transportation Officials.

8. *Alaska Standard Specifications for Highway Construction*, 2020 Edition.

9. Zobel, Robert S.; Jirsa, James O.; *Performance of Strand Splice Repairs in Prestressed Concrete Bridges*, PCI Journal, Volume 43, Issue 6, Nov/Dec 1998.