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PART 2 – REHABILITATION SELECTION

REHABILITATION SELECTION

Part 2 of this Manual provides guidelines for selecting methods and determining the appropriate strategy for repairing, rehabilitating, or replacing structural components. The selection process requires careful consideration of technical, economic, and practical factors to ensure an optimal solution. The guiding principles are outlined in Section P2-1, considerations for each component is presented in P2-2, and various rehabilitation treatments are described in Section P2-3.

P2-1 Principles for Rehabilitation Decision-Making

P2-1.1 Review of Data

Before developing a proposed scope of work, the Engineer shall review all available data on the structure to understand its condition and history. As a minimum, the Engineer shall conduct a review of the following:

- **Drawings**
  Design, as-built, and shop drawings to obtain dimensions, details, material properties, and features that may constrain design and construction, such as embedded or attached utilities, clearance requirements, and roadway widths for staging.

- **Inspection and Investigation Reports**
  Inspection records, condition surveys, and other relevant reports to assess the type, location, extent and history of deterioration, evaluate the performance of previous repairs and rehabilitation efforts, and identify potential risks.

P2-1.2 Rehabilitation Strategy

P2-1.2.1 Background

Ministry policy toward asset management has evolved significantly over the years. Beginning in the early 1970s, the Ministry began to formalize its approach to bridge rehabilitation. During this period, many bridges originally constructed with exposed decks were retrofitted with concrete overlays (including with latex-modified concrete), waterproofing membranes and asphalt wearing surfaces. By the 1980s, many mid-century bridges were experiencing severe corrosion, and their life expectancy was often limited to approximately 40 years.

In the early 1990s, rehabilitation spending was limited, and bridge work shifted towards minimal holding strategies with shorter anticipated service lives. Despite constrained budgets, concrete removal policy continued to target both delaminated and unsound concrete as well as sound concrete in areas of high corrosion potential. From the late 1990s through early 2000s, the Ministry followed an asset management approach guided by the *Prioritised Contract Content (PCC) Guidelines* and coping strategies aimed at extending the service life of the aging inventory, with a preference for rehabilitation over replacement. This approach resulted in numerous comprehensive major bridge rehabilitations that were anticipated to extend their service life by 25 to 50 years, although these projects were labour-intensive and often require construction duration similar to full bridge replacements.

In 2008, government policy changed to prioritize direct spending on roads and bridges. During this period, the focus was on maintaining the existing network and replacing structural components on a like-for-like basis, with improvements made only in exceptional cases. In 2015,

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the Ministry issued policy memo (BO-2015-01), *Strategic Management of Ontario's Provincial Structures from 2015 to 2018,* which emphasized minimizing complex bridge rehabilitations and prioritizing minor rehabilitation strategies or full structure replacement. As a result, major complex bridge rehabilitations were discouraged, and component replacements were generally limited to elements in poor condition.

By 2018, a substantial portion of the Ministry's structural assets in poor condition had been addressed. From 2018 to the present, the Ministry's approach has shifted back toward more comprehensive rehabilitations, often including semi-integral conversions as well as barrier and bridge deck overhang replacements.

Rehabilitation can be broadly classified into three main approaches: preservation management, structural rehabilitation and structure replacement.

P2-1.2.2 Preservation Management

Preservation management refers to the proactive application of low-cost, low-disruption treatments that slow the onset of deterioration and extend service life before significant defects develop.

A preservation management strategy shall be applied to extend the longevity of bridges with decks originally protected with hot-applied rubberized waterproofing, protection boards, and asphalt riding surfaces, and that currently show minimal signs of deterioration. These bridges were constructed from the late 1970s onward. These bridges correspond to those described in Part 1, for which a Level 1 condition assessment, the most basic level of evaluation, is recommended prior to their first rehabilitation (Refer to Section P1-2.1 Level 1 Condition Assessment).

While many of these structures do not present immediate needs based on their Bridge Condition Index (BCI), effective preservation management delays the onset of deterioration through treatments that can be implemented quickly, require minimal traffic disruption, are cost-effective, and are relatively simple to design. Preservation contracts should target construction durations of less than one month and aim to achieve a minimum of 15 years before additional work is required.

Preservation management strategies should generally be limited to the following treatments:

- Replacing hot-applied rubberized waterproofing every 30-35 years. Refer to Section P2-3.11 Waterproofing.
- Applying surface sealer to concrete surfaces continuously exposed to chlorides, including barrier walls, soffit of cantilever overhangs, and substructure components within the splash zones. Refer to Section P2-3.7 Concrete Coating.
- Replacing expansion joint seals. And replacing expansion joints if their seals cannot be readily replaced. Refer to Section P2-3.9 Expansion Joints.
- Zone painting of steel structures.
- Replacing corroded metal railing elements.
- Pressure-washing ACR steel bridges to remove loose, flaky and unstable patina. Refer to Section P2-3.14.3.5 Repairing Deteriorated ACR Steel Patina.

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P2-1.2.3 Structural Rehabilitation

Rehabilitation is almost always technically feasible, but it is not always economical. Bridge rehabilitation strategies range from short-term holding strategies that extend service life by 10 to 15 years, to major rehabilitations that extend service life by 30 years or more. Major rehabilitations shall consider extending the bridge's service life beyond its original design life, based on an assessment of its current condition and anticipated future use.

The economic viability of rehabilitation depends on several factors, including the structure's design details, construction quality, maintenance history, and the traffic staging impacts associated with the work. In many cases, fixed ancillary costs related to highway and staging requirements represent a significant portion of the overall project cost. Engineering effort and costs for complex rehabilitation projects are similarly high. Therefore, a thorough evaluation of total cost, impacts, and effort must be undertaken when selecting the appropriate rehabilitation strategy.

P2-1.2.3.1 Holding Strategies

Holding strategies are generally less economically effective for managing bridge assets. They should be avoided wherever possible through advanced planning and by considering more durable rehabilitation approaches or full replacement. Designers should recognize the decreasing effectiveness of repair techniques as bridges age; the service life of rehabilitation treatments typically diminishes on older structures. Subsequent rehabilitations inherently yield shorter service lives, and this reduction must be accounted for when evaluating options for structures with progressing deterioration. For instance, patch repairs completed as part of a second rehabilitation on a structure already exhibiting high chloride contamination should not be expected to extend its service life by more than 15 years before further intervention is required.

P2-1.2.3.2 Major Rehabilitations

The scope shall focus on the bridge components that require intervention based on their condition and structural performance. Components shall be repaired or replaced to meet the targeted service life of the overall rehabilitation.

Conversion to semi-integral abutment details should be limited to bridges where the intended service life extension is 40 years or more. For bridges constructed after 2012 with continuous superstructures and deck-end components reinforced with stainless steel, semi-integral conversion is not recommended. In these cases, other corrosion protection methods should be considered to preserve the substructure, and deck ends against potential future joint leakage. When converting to semi-integral, evaluate proactive retrofit or partial replacement of surrounding elements to capitalize on access and ensuring that the targeted service life of the elements is aligned.

P2-1.2.3.3 Superstructure or Full Structure Replacement

Bridges

Replacement shall be considered for bridges constructed prior to 1973 that exhibit significant deterioration and require major repair. Deck protection systems on pre-1973 bridges were not

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consistent; some decks were left exposed, while others were waterproofed using systems with varying degrees of effectiveness.

A replacement strategy offers several benefits, including reduced risk associated with unknown conditions in existing structures, a longer service life for the asset, fewer total traffic disruptions over the life cycle, improved durability through modern design provisions, opportunities for roadway geometric improvements, and potential functional enhancements. The timing of replacement shall be based on operational needs and available funding.

Superstructure replacement shall be considered where the foundations and substructure elements are in good condition. Investigations to confirm the condition of buried components should be included in engineering assignments where appropriate. Substructure rehabilitation and foundation strengthening are often sustainable and effective measures to extend their service life to match that of new superstructures.

Deck replacements shall only be carried out with approval from the Head of the Regional Structural Section. Superstructure replacement is preferred when thin-slab decks require replacement.

Structural Culverts

For structural culverts, replacement shall be considered as a treatment to reduce future intervention. In many cases, the cost of the culvert structure itself is relatively small compared with the costs associated with site investigations, engineering, temporary works, and highway staging. As a result, the longevity gained through full replacement may outweigh the marginal additional cost compared with rehabilitation. However, reinforced concrete culverts can generally tolerate extended periods of deterioration before replacement becomes warranted. Engineering judgement shall be applied to ensure ultimate limit state (ULS) load cases are satisfied, including using refined analysis methods where reinforced concrete culverts exhibit advancing deterioration.

For soil-metal structures, treatment selection shall include consideration of trenchless lining methods whenever open-cut replacement would result in significant disruption. Refer to Section P2-3.16 Liners for Structural Culverts for information related to structural culvert liners.

P2-1.3 Design and Service Life

The following definitions apply, adapted from *CSA S6-25 CBHDC*:

- **Design life**: The notional period used in the derivation of design loads and load combinations to achieve the required level of reliability and functional performance. Design life may be independent of actual service life.
- **Service life**: The actual period of time during which a structure or component remains fit for its intended use without major repair or replacement of non-replaceable components.
- **Design service life**: The planned and specified service life established at the time of design.

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P2-1.3.1 Service Life Policy

Rehabilitation strategies shall align with the remaining service life of the structure. Where a structure no longer meets current requirements for geometry or load capacity, or exhibits deficiencies that limit its service life, replacement shall be considered. In such cases, interim rehabilitation may be undertaken to maintain serviceability until replacement.

A comparison matrix of rehabilitation and replacement alternatives shall be conducted using a parametric evaluation framework. Each alternative shall be assessed against quantifiable criteria, including but not limited to:

- Remaining expected service life of components (replaceable and non-replaceable, in years)
- Future corridor requirements, if applicable
- Construction duration (in months)
- Traffic staging considerations (temporary lane widths, delay or queuing)
- Site access constraints
- Utility relocation costs ($)
- Environmental allowable work periods
- Constructability
- Construction cost (structural and grading), inclusive of temporary measures (traffic staging, temporary protection systems, dewatering etc.).

The evaluation shall include a detailed comparison of project impacts to ensure an objective, and data-driven comparison.

Typical service lives of replaceable components provided in *CSA S6.1-25* Commentary Clause 2.5.2.4 shall be used for guidance and supplemented by testing, judgement and consideration of site-specific conditions.

Where deterioration is minor, rehabilitation methods shall be evaluated for cost-effectiveness. Where appropriate, deferred intervention or preventative maintenance may be selected, provided the consequences of inaction are assessed and found acceptable. Conversely, proactive replacement of components may be undertaken to better align remaining service lives across the structure.

Where the timing of replacement is uncertain, rehabilitation strategies with an expected service life extending beyond the anticipated replacement date should be considered to reduce the likelihood of repeated interventions prior to replacement. The original design service life for a structure shall not be used as a target for the end of actual service life. The condition, performance, details, and materials shall be duly considered at the time of intervention, and a predicted remaining service life shall be estimated for the rehabilitation strategies considered. Wherever viable, and extension of service life beyond its original design service life shall be studied.

Major structures (spans exceeding 300 m) may warrant extended service life targets due to the significant challenges associated with replacement. Such assets shall be evaluated on a case-by-case basis and shall not be treated in the same manner as typical bridge structures.

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P2-1.3.2 Rationale (Informative)

In the MTO bridge rehabilitation context, lifecycle cost analysis (LCCA) has predominately been used to justify investing in rehabilitation rather than opting for immediate bridge replacement. By deferring major expenditures, such as replacement costs, LCCA enables for more rehabilitation work in the short-term. This approach is particularly beneficial in financially constrained environments, as it optimizes network performance by spreading investments across existing infrastructure.

Overall, LCCA is a valuable decision-making tool within asset management, providing direct comparisons between alternatives that provide similar performance. LCCA according to the requirement of the Ministry's Structural Financial Analysis Manual (1990) focuses primarily on financial costs. However, it is essential to account for other critical factors in the decision-making process, such as durability, long-term maintenance, traffic impacts, safety benefits, and more.

Opting for more durable solutions upfront might incur higher initial costs which LCCA may not fully justify. LCCA tend to favour solutions with replaceable components that defer costs to the future. Therefore, while LCCA provides valuable insight, it is important to recognize that it is just one component in a broader decision-making framework.

Advancements in materials, design practices, corrosion protection systems, and construction methods have significantly improved durability of modern bridges. As a result, rehabilitation intervals have extended toward approximately 35 to 40 years, with many modern bridges now anticipating only preventative maintenance within their service life, such as replacement of waterproofing systems, expansion joints, and in some cases, bearings.

Observed performance indicates that bridges constructed with modern corrosion protection systems and appropriate detailing exhibit improved durability and lower rehabilitation needs. The increased use of integral bridge designs is expected to further reduce long-term maintenance requirements.

A service-life-based approach supports more effective allocation of rehabilitation resources by aligning intervention strategies with actual performance, anticipated future use, and replacement constraints, particularly for large, complex, or difficult-to-replace structures.

P2-1.4 Considerations

P2-1.4.1 Urgent Safety and Operational Concerns

When immediate operational or safety issues are identified, timely action is essential to mitigate risks and prevent further deterioration that could lead to accidents or unplanned closures. In these situations, emergency repairs should be undertaken to maintain the structure in a safe and serviceable condition. These interim measures provide operational continuity until a comprehensive rehabilitation can be delivered to address the underlying deficiencies.

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P2-1.4.2 Network and Program Alignment

P2-1.4.2.1 Importance of Structure

The importance of a structure is influenced by factors such as traffic volume, the role of the highway within the broader network, the availability of alternative routes, and in some cases, the size and complexity of the bridge. These considerations affect both the operational impact of rehabilitation activities and the overall cost of the work. Structures that serve critical functions or lack viable detour options may warrant enhanced protective treatments to extend their service life, reduce future interventions, and better support long-term network performance.

P2-1.4.2.2 Highway Construction Program and Long-Term Corridor Plans

The timing of structure rehabilitations is closely tied to the broader highway construction program. Bridge rehabilitations are often incorporated into highway improvement contracts. It is imperative to investigate future highway plans and other proposed structure contracts in the vicinity. The outlook for the ultimate cross section of the facilities on and below the bridge shall be considered in the preliminary stages of the design.

Where future highway realignment or geometric improvements are planned, the work on the affected structures should be minimalized, with a focus on maintaining them in service until the highway improvements are carried out. However, delaying rehabilitation work may lead to further deterioration and a more costly rehabilitation in the future and needs to be accounted for.

Conversely, it may be justifiable to include rehabilitation of nearby structures that don't urgently require it in a highway rehabilitation contract to capitalize on the economies of scale. Efforts should be made to ensure that the structure rehabilitation aligns with other activities within the contract.

P2-1.4.2.3 Uniform Approach

To ensure consistent rehabilitation decision-making, apply a uniform strategy to bridges that form a defined bridge family (e.g., structures within the same corridor or of similar age and type). Where feasible, deliver rehabilitation of bridge families under a single contract to improve cost-effectiveness and operational efficiency. Standardizing rehabilitation approaches across a bridge family can generate economies of scale and support alignment of future maintenance and intervention schedules.

P2-1.4.3 Structural Capacity, Condition and Standards

P2-1.4.3.1 Load Carrying Capacity

Bridge elements may be required to carry additional loads during or after rehabilitation, from new loads applied directly to the component or through redistribution of loads from other components. Rehabilitation may result in changes to the behaviour or articulation of the bridge such as from freeing or fixing restraints or bearings at supports. These factors can result in loads and stresses in components that were not accounted for in the original design of the bridge.

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When the loading or load-carrying capacity of a structure is affected by defects, deterioration, or rehabilitation methods, an evaluation must be carried out to check that the bridge and its components can adequately carry the loads before, during and after rehabilitation, maintaining stability throughout. All significant changes in loads and capacity must be accounted for.

P2-1.4.3.2 Material Defects and Deteriorations

The type, extent, and location of defects and deterioration are captured and recorded during the condition assessment stage of projects. The underlying causes of defects and deterioration must be established to select the most appropriate method and strategy for rehabilitation. When the contributing causes of defects are not eliminated and risks remain for similar deterioration in the future, the rehabilitation should incorporate strategies to mitigate and reduce those risks. The extent and location of the deterioration influences and can also limit the choices of repair and rehabilitation methods.

P2-1.4.3.3 Obsolete Design Standards

Bridges have been designed and constructed over the decades according to different loads and standards. Consequently, some existing structures may meet requirements that differ those currently in effect. When the load capacity or other aspects of an existing structure are evaluated, the assessment may show that it does not fully comply with the design criteria required for new construction. In such cases, the Owner must determine whether the structure should be upgraded to meet current standards.

In some situations, it may not be necessary for an existing bridge to satisfy all loading requirements specified in the current *CSA S6 CHBDC* for new bridges. Depending on the intended use and remaining service life, a bridge may be rehabilitated for reduced load levels. This approach is typically suitable for structures on low-volume roadways serving light local traffic, or for bridges planned for replacement soon. In these cases, rehabilitation may be carried out according to the reduced load criteria in Section 15 (Rehabilitation and repairs) of the *CSA S6 CHBDC*, with the bridge posted and signed accordingly to reflect its restricted use.

Before adopting any reduced standards in a rehabilitation design, potential impacts on public safety and future operations must be thoroughly assessed. The design must ensure that the bridge retains adequate capacity for emergency vehicles and essential road maintenance equipment.

P2-1.4.3.4 Need for Foundation Evaluation

A foundation evaluation must be carried out in each of the following circumstances:

1. If there is evidence of existing foundation problems (e.g., significant movements or settlements)
2. If a bridge rehabilitation is being undertaken and the total dead load on the bridge, compared to the original design, is changing (increasing or decreasing) by more than 10%
3. If the articulation of the bridge is changed, (e.g., sliding bearings changed to fixed, simple span changed to continuous, etc.)
4. If the existing foundation is being widened
5. If the bridge superstructure is being widened by more than 500 mm

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6. If staging is being implemented that may place unusual loads on the foundation

The Foundations and Geotechnical Systems section of the *CSA S6 CHBDC* addresses the design of foundations, but the Evaluation section specifically excludes the evaluation of foundations and instead requires using design provisions to determine the capacity of a foundation. For rehabilitation work this may result in overly conservative designs and lead to costly strengthening measures. Foundation elements may be evaluated with a lower Reliability Index if there are adequate ductility and redundancy in the load carrying system.

Additionally:

1. Where evaluation is required and adequate information does not exist on the soil conditions, a geotechnical investigation must be conducted to determine the required subsurface conditions.
   a. The degree of investigation required must be determined in consultation with the Foundation Design Section.

2. For structure widening, the evaluation must consider the change in surcharge load and the interaction of the existing and new foundations in terms of differential settlements and movements.
   a. For minimal substructure widening (less than 5 m), the widened portion of the foundation can be designed in accordance with *CSA S6-25 CHBDC* Clause 15.15.2.2, or a lower level as specified in Bullet 4, below.
   b. For new structures and widenings (greater than 5 m), the widened foundation must be designed to the current *CSA S6 CHBDC* standards of Section 6, unless the provisions from the Low Volume Bridge Guidelines in Division 1 of the *Structural Manual* apply.

3. Where a foundation evaluation indicates a deficiency, the analysis, assumptions, and methodology must be refined prior to committing to foundation strengthening or a structure replacement due to foundation capacity concerns.

4. If a foundation evaluation indicates a deficiency but the cost to retrofit is prohibitive, the existing foundation may be accepted by the Regional Structural Head, in consultation with the Structures Office. Consideration must be given to the performance of the existing foundation, the degree of reduction of safety factor, the consequences of failure, a cost analysis, and the remaining service life of the bridge.

P2-1.4.4 Site Context, Resources and Logistics

P2-1.4.4.1 Geography and Remote Areas

Bridge construction in remote areas presents additional logistical challenges and a shorter construction season. When defining the scope of work, it shall reflect and address geographical limitations. Access to skilled labour and equipment may be limited, and longer delivery routes and times shall be considered for the provision of construction materials and equipment to these sites. Ready-mix concrete plant delivery distances shall be confirmed in design to reflect concrete discharge time requirements. Where ready-mix is not feasible, mobile batch plants shall be considered in combination with cast-in-place quantities required on site. Preblended bulk bag mix concrete shall be considered the least preferred option and shall be supplied by a Ministry

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approved supplier when applicable. Prefabrication is preferred when it is able to reduce the duration and complex of the work on-site. Seasonal load restrictions on routes shall be considered in planning of the delivery of construction materials, equipment and structure components.

P2-1.4.4.2 Local Expertise

Consideration must be given to the expertise of local contractors and available construction equipment. Rehabilitation methods and materials that require a specialized contractor from outside the area may not be cost-effective for small contracts. This is also true for specialized equipment. In such cases, alternative rehabilitation methods that can be carried out by local contractors should be considered.

P2-1.4.5 Construction Strategy

P2-1.4.5.1 Staged Construction

The most recent version of the Ministry's Guidelines for Staged Construction of Bridges outlines issues that must be considered, when designing bridge widenings and bridges built or rehabilitated in multiple stages. The guidelines discuss various methods to stage thick and thin slab bridges, closure strips, shrinkage induced lateral movement, mitigation of live load effects, and temporary concrete barriers.

Rehabilitations are significantly influenced by traffic staging and access requirements, as these factors dictate both the feasibility and methodology of the repairs. Some repairs, such as silica fume shotcrete, requires continuous access to the work area during curing. If continuous access cannot be maintained, alternative methods or materials may need to be considered. For soffit repairs, vertical clearance requirements must be considered both during construction and in the rehabilitated condition.

P2-1.4.5.2 Avoid Labour Intensive Details

To reduce construction costs and shorten project duration, it is worthwhile to explore ways to minimize labour-intensive details in design. A common example is to reduce the need for concrete removal using chipping hammers, especially when reconstructing the deck edge cantilever along an entire bridge. While not always feasible, detailing full removals using saw cutting or methods such as hydro-demolition could be considered (refer to Section P2-3.1 Concrete Removal).

P2-1.4.6 Heritage and Aesthetics

P2-1.4.6.1 Heritage Bridges

The process for identifying, evaluating, and listing bridges of cultural heritage value, conservation options, and the management of heritage bridges is presented in the most recent version of the Ministry's Ontario Heritage Bridge Guidelines.

P2-1.4.6.2 Aesthetics

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The most recent version of the Ministry's Aesthetic Guidelines for Bridges provides helpful information on numerous principles of bridge aesthetics including general appearance and aesthetic treatments. These guidelines are primarily aimed at MTO *workhorse* bridges, that is, short to medium span bridges with a main span of 80 m or less. The guidelines are supplemented by Bridge Office #2018-03 memorandum, which clarifies procedures and responsibilities of the Ministry Bridge Aesthetics Evaluation Group (MBAEG) and Regional Structural Offices.

P2-2 Rehabilitation Considerations by Structural Element

P2-2.1 Superstructure Components

P2-2.1.1 Bridge Deck

Several flowcharts were included in the 2007 Structure Rehabilitation Manual under Appendix 2.C to guide decision-making for bridge deck rehabilitation and replacement. However, with the standardization of deck waterproofing practices, improvements in detailing, and the evolution of the Ministry's corrosion protection policy, the likelihood that bridge decks will require full replacement has been significantly reduced. Modern decks, properly waterproofed and constructed in accordance with the Ministry's corrosion protection requirements, are generally expected to remain serviceable through targeted repairs over their service life rather than requiring premature replacement. The updated flowchart in Figure 9 does not contradict the 2007 versions; rather, it is tailored to the characteristics of the current bridge inventory and emphasizes the practical choice between patch repairs and overlays. It is intended as a guidance tool, not a rigid prescription, allowing engineers to apply judgment and adapt the process to project-specific conditions and findings.

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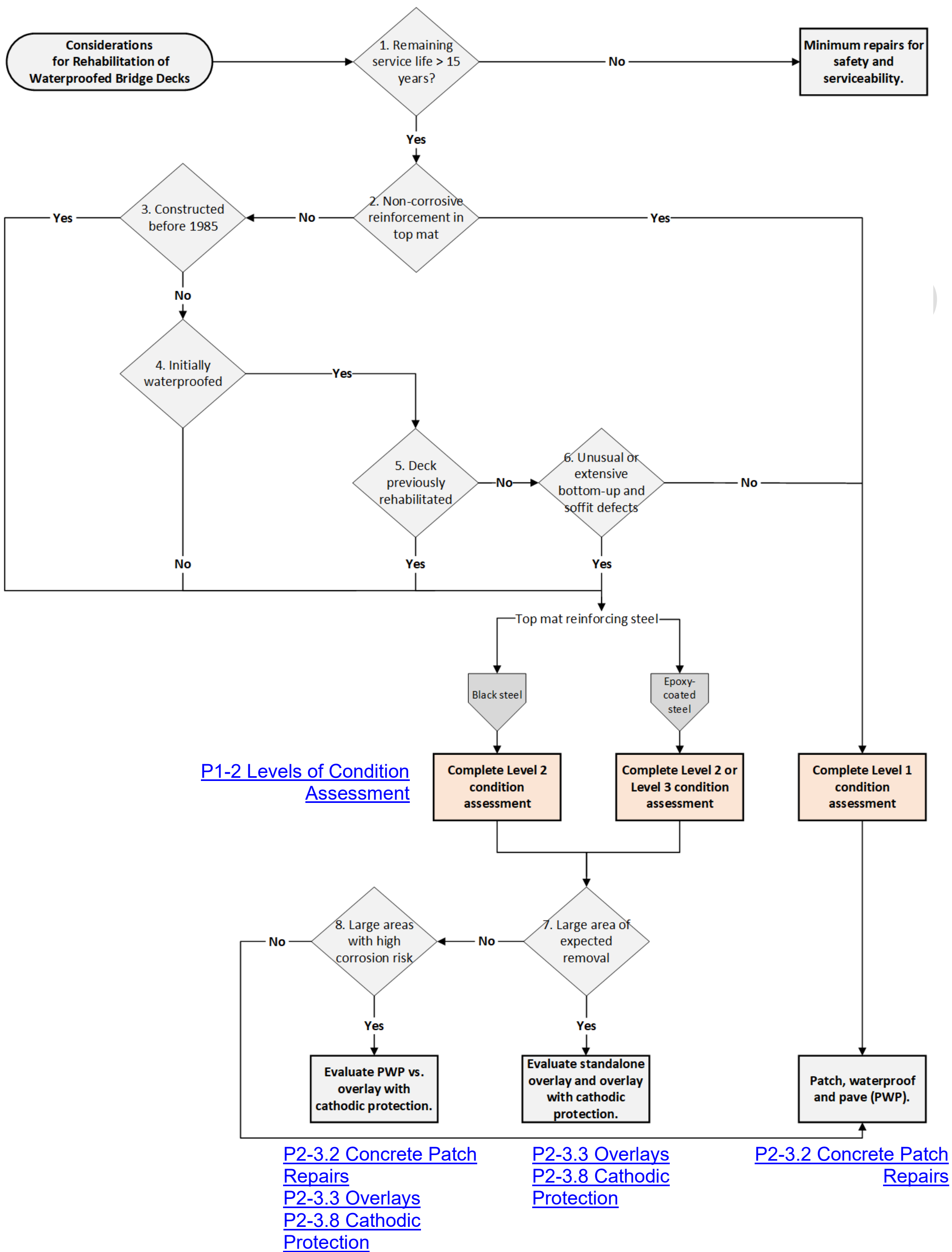

Figure 9 - Considerations for Selecting Rehabilitation Strategy for Bridge Decks.

When the bridge is **expected to be replaced within 15 years (◇1**, *corresponds to the numbered decision points in the flowchart***)**, work shall be minimized and focused on maintaining the safety and operation of the bridge.

Bridge decks with non-corrosive reinforcement, such as stainless or FRP, in the top mat (◇2) are typically newer structures undergoing their first rehabilitation. These decks were originally constructed with waterproofing system and reinforcement materials that do not corrode, resulting in a minimal risk of chloride-induced deterioration. Consequently, future work is expected to consist primarily of limited, localized patch repairs.

In the flowchart, steps **◇3** through **◇8** form a connected sequence of decisions. These steps, covering construction era, original waterproofing, prior rehabilitation history, and the presence of unusual or extensive deterioration, work together to determine the appropriate level of condition assessment and subsequent rehabilitation approach. Because these steps are interdependent, they are discussed collectively in the sections below.

Table 18 summarizes the characteristics of MTO bridges undergoing their first and second rehabilitations including their scope of deck repairs and chloride levels. The data is collected from over 100 bridges rehabilitated between 2000 to 2020. The table provides a reference baseline for comparison with the typical MTO bridge.

Table 18 – Comparison of Bridges Undergoing First and Second Rehabilitations.

		1st Rehabilitation	2nd Rehabilitation
	Age at time of construction	35 +/- 4	52 +/- 7
Bridge deck scope of work	PWP	95%	65%
	Overlay	5%	25%
	Replace	0%	5%
	Percentage of concrete cores with defects	5%	24%
Average chloride profile (removed background Cl-), %	0 - 10 mm	0.026	0.134
	20 - 30 mm	0.019	0.107
	40 - 50 mm	0.013	0.075
	60 - 70 mm	0.011	0.041

Bridges undergoing their first rehabilitation, typically those **constructed after 1985** (**◇3) and protected with the standard waterproofing system (◇4)** and an asphalt wearing surface, generally have decks in good condition. Most of these decks exhibit minimal signs of deterioration or chloride contamination beyond the top surface. An overwhelming 95% of these decks were **patched, waterproofed, and paved as their first rehabilitation (◇5**). As supported by the data, the standardization of the bridge deck waterproofing system has led to a slower, more consistent, deterioration of bridge decks when compared to previous decades. As a result, the first rehabilitation of waterproofed bridge decks primarily emphasizes preventative maintenance.

Bridges undergoing their **second rehabilitation (◇5)** are generally constructed before 1985 **(◇3)**. The changes in design and construction practices from the 1960s to 1980s are evident in their deck repairs; they are less uniform compared to newer bridges undergoing their first rehabilitation. While patching remains the most prevalent deck repair strategy, nearly a quarter of the decks were overlaid, and few were replaced.

The differences become clearer when categorizing these decks into decks that were patched and those that were overlaid, as presented in Table 19. It is important to recognize that chloride profile of the bridge deck is an important indicator of **corrosion risk (◇8)** in bridge decks, but it should not be assessed in isolation. It must be evaluated alongside observed defects such as cracking, delamination, spalling, and rust staining, as well as other key factors including concrete permeability, concrete cover, reinforcement type, moisture exposure, and the performance of existing waterproofing systems. In addition to these condition indicators, the choice of rehabilitation treatment should also consider the **expected extent of concrete removals (◇7)**, as this directly affects whether localized patch repairs or a full overlay is the more practical and economical option.

Table 19 – Characteristics of Bridges Undergoing Their Second Rehabilitation.

		2nd Rehabilitation	
		PWP	Overlay
Percentage of concrete cores with defects		17%	35%
Average chloride profile (removed background Cl-), %	0 - 10 mm	0.105	0.195
	20 - 30 mm	0.082	0.154
	40 - 50 mm	0.058	0.106
	60 - 70 mm	0.034	0.053

As summarized in Table 19, bridge decks rehabilitated with overlays exhibit greater levels of deterioration and chloride contamination compared to decks that were rehabilitated through patch repairs. Overlays use more concrete and are generally more expensive, on a total cost basis, than patch repairs. However, where there are significant areas of deterioration, their unit costs become more comparable. The ‘breakeven’ point varies and is influenced by factors like the deck area, staging requirements, availability of concrete, labour, and equipment resources.

Overlays are also a risk mitigation strategy, especially for bridge decks with **low concrete cover**, **extensive areas of chloride contamination, and/or high corrosion potential (◇8)** that may not have yet manifested as visible deterioration. Overlays provide additional concrete cover to slow down the ingress of chlorides and may potentially cause upward migration of chlorides from the original concrete into the new concrete.

Overlays add extra dead load to the structure, some bridges may lack sufficient capacity for the additional weight. The Ministry has constructed exposed overlays to replace existing waterproofing and asphalt wearing surface, to minimize additional load imposed on the structure. Exposed overlays are constructed using low permeable concrete such as latex modified concrete. Exposed overlays have limitations; cracks in the bridge deck must be repaired beforehand, otherwise they tend to reflect through. Routine maintenance, such as patching the concrete wearing surface, takes longer compared to similar repairs on asphalt and may not be suitable for high-volume roads. Different types of overlays are described in Section P2-3.3 Overlays.

It is important to reiterate that the assessment levels outlined in P1-2 Levels of Condition Assessment represent the minimum required scope of investigation. **When unusual or extensive deterioration is present or when essential information is missing or uncertain (◇7)**, the engineer shall augment these minimum requirements with any additional testing or evaluation methods needed to support a well-informed rehabilitation decision.

On projects considering exposed overlay or cathodic protection, consult the Regional Structure Office and Structures Office as early as possible.

P2-2.1.2 Bridge Soffit

The options for repairing deteriorated areas on the deck soffit are usually limited to concrete patch repairs using form and pump, or shotcrete (refer to Section P2-3.2 Concrete Patch Repairs). Full-depth repairs should be carried out when soffit removals coincide with removals for the deck top.

P2-2.1.3 Bridge Deck Fascia

The options for repairing the deck fascia include concrete patch repairs or refacing the entire concrete fascia (refer to Section P2-3.2 Concrete Patch Repairs and Section P2-3.4 Refacing and Jacketing). Refacing or reconstructing the deck edge should be considered, especially over travelled lanes, when there is significant deterioration along the fascia.

When rehabilitation work includes replacing the concrete barrier walls, it may be appropriate to replace the entire deck edge to the centreline of the exterior beam, if:

- the fascia and/or soffit along the deck edge is in poor condition, or
- significant amount of concrete removal is expected along the existing curb line on a cantilevered thin deck slab.

The deck fascia is one of the most visually prominent features for users travelling underneath. The detailing should aim to either maintain the fascia in full or replace it entirely, as partial replacement often creates a noticeable and visually displeasing contrast between the existing and new concrete.

P2-2.1.4 Barriers (Railings Systems and Walls)

This section replaces Bridge Office Memorandum 2019-01 dated January 24, 2019.

P2-2.1.4.1 Background

Throughout the 1980s, significant research was conducted on barrier performance in crash tests through the National Cooperative Highway Research Program (NCHRP) and results were published in Report 230 (1981) and Report 350 (1993). At that time, MTO Structures Office had already started to implement the concept of crash tested barriers in its standards.

With the release of the 3rd edition of the OHBDC in 1993, bridge barriers were required to conform to crash tested standards based on appropriate “performance levels” (i.e., PL-1, PL-2 and PL-3) determined by a site-specific exposure index and warrant.

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When *CSA S6-00 CHBDC* replaced the OHBDC, it maintained the requirement for crash tested barrier performance levels but revised the design loads for barrier anchorage in deck overhangs, including a significant increase for PL-3 railings.

With the release of *CHBDC S6-14*, these performance levels (PL) were renamed test levels (TL) to align with NCHRP Report 350 terminology. It was primarily a nomenclature change with little effect on the design loads.

The FHWA first introduced the *Manual for Assessing Safety Hardware (MASH)* in 2009, followed by a second edition in 2016. MASH increased crash-test loads, most significantly for TL-4, with smaller increases for other test levels. These requirements were incorporated into *CSA S6-19 and S6-25 CHBDC*. However, through MTO Exception to *CSA S6-25 CHBDC*, traffic barriers may be qualified by using either the previous NCHRP Report 350 criteria or the newer MASH requirements, both remain acceptable for compliance.

A summary of the code and MTO's historical approach to barriers is presented in Figure 10.

	Bridge Code	**Code Barrier Philosophy**	**MTO Barrier Standard Philosophy**
2020	CHBDC 2025	NCHRP 350 or MASH	NCHRP 350 (TL terminology, with some modifications) or MASH
	CHBDC 2019		NCHRP 350 (TL terminology)
	CHBDC 2014	NCHRP 350	
2010	CHBDC 2006	NCHRP 230	NCHRP 350 (retained PL terminology)
2000	CHBDC 2000		
	OHBDC 1991		NCHRP 350 (some modifications, retained PL terminology)
1990	OHBDC 1983	Unit Load	NCHRP 230 (some modifications)
1980	OHBDC 1979		Unit Load (generally smooth barriers)
1970 1960	AASHTO		Unit Load (generally post and railing panels)

Figure 10 – The Code and MTO's Historical Approach to Barriers.

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P2-2.1.4.2 Barrier Categories and Equivalent Test Levels

- Barriers with design drawings indicating PL-1, PL-2, or PL-3 shall be considered adequate for TL-2, TL-4, and TL-5 respectively.

- Barriers with an F-shape, New Jersey shape or other safety shape profiles that are 1025 mm or taller are considered adequate for TL-5. These barriers typically have a lower 300 mm section sloped at about 55° with an upper slope of about 5 to 10°. Despite numerous MTO design variations from the 1980s through the early 2000s, in-service performance has been consistently similar across these types.

- Barriers with an F-shape, New Jersey shape or other safety shape profiles that is 800 mm or taller are considered adequate for TL-4. These have similar slopes as described above but with a shorter upper portion.

- Solid concrete MTO standard barrier designs with smooth continuous face 800 mm or taller are considered adequate for TL-4, while those less than 800 mm are considered adequate for TL-2 barriers.

- Solid concrete MTO standard barrier designs without a smooth face, such as post and discrete rails, or post and rail panels, whether steel or concrete, are considered adequate for TL-1.

P2-2.1.4.3 Crash Test Considerations for Replacing & Rehabilitating Barriers

Barrier adequacy shall be reviewed against current *CSA S6 CHBDC* requirements whenever a bridge is being rehabilitated or repaired, except as noted below, or as directed by the MTO Structural Section Head.

- **Holding Strategies (≤10 years)**
  A review is not required unless the work involves more than minimal local barrier repairs.

- **Substructure-Only or Minor Barrier/Sidewalk Repairs**
  A review is not required when the rehabilitation scope is limited to substructure work or minor barrier / sidewalk repairs.

- **Deck Patching + Waterproofing Replacement**
  A review is required when the rehabilitation scope involves deck patching and replacing waterproofing.

- **Overlays**
  A review is required, including minimum barrier height requirements, when the rehabilitation scope involves an overlay or results in a net increase in thickness.

Where a review determines that existing barriers do not meet current *CSA S6 CHBDC* requirements, the barriers shall be replaced. Engineering judgement shall be used to determine barrier adequacy and whether barrier replacement is required on low-volume roads and single-lane truss structures, based on the rehabilitation scope.

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P2-2.1.4.4 Rehabilitation Options for Barriers

If the existing concrete barrier wall or parapet wall is being maintained, the following rehabilitation options shall be considered: concrete patching (Section P2-3.2 Concrete Patch Repairs), concrete refacing (Section P2-3.4 Refacing and Jacketing) or applying a concrete sealer (Section P2-3.7 Concrete Coating).

Refacing shall be applied along the full length of the affected barrier face. Barriers with partial-height back-face vertical reinforcement or with discontinuous or nominally lapped longitudinal reinforcement at control joints, details that are common in structures built in the 1980s and 1990s, shall not be refaced. Where such conditions exist and refacing would otherwise be warranted, the barrier shall be replaced.

Preference should be given to replacement over refacing when construction schedule impacts are acceptable to the Region, and when replacement aligns with the expected service life of the entire deck.

Barriers with deficient performance, as determined under Section P2-2.1.4.2 Barrier Categories and Equivalent Test Levels, shall be replaced. Barrier replacement may also be undertaken, subject to approval by the MTO Structural Section Head, where site-specific collision history, material condition, staging constraints, or roadway operational improvements support full barrier replacement.

P2-2.1.4.5 Additional Considerations

Structural Capacity Review

When a barrier is replaced, the structural adequacy of the connected components, bridge deck, wingwalls, or retaining wall etc., shall be assessed to confirm their capacity to support the new barrier and to verify sufficient space for required reinforcing details.

Requirements for Evaluating Deck Cantilevers, Wingwalls, and Retaining Walls

When assessing capacity for new barrier loads:

- Deck cantilevers, wingwalls, and retaining walls shall be evaluated in accordance with the *CSA S6 CHBDC* and MTO Exceptions; evaluation shall not exceed TL-4 loading. The Evaluation section of *CSA S6 CHBDC* and a target reliability index of 2.75 may be used.
- For retaining wall overturning and sliding under vehicle collision loads, a load factor of 1.0 may be applied to horizontal earth pressure and related dead loads, combined with the appropriate live-load impact factor at ULS.
- Where a refined analysis is used, moment redistribution may be applied over a width greater than that derived from elastic analysis.
- Live-load and compaction surcharges may be omitted when assessing retaining walls for barrier impact loads.

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Partial-Height Back-Face Vertical Reinforcing Steel

Barriers with partial-height back-face vertical reinforcement have unreinforced concrete along the top of the back face, which may be more prone to spalling under vehicle impact than barriers with full-height reinforcement. Although this increased risk has not been quantified, concerns may arise where there is a history of collisions and a possibility that spalled concrete could fall onto the roadway below during an impact. In such cases, where full replacement is not warranted, FRP wrapping along the back face, limited to sections directly over live lanes, may be considered as a retrofit specifically to contain any concrete that may spall if struck. In most situations, this retrofit is not required.

One-Sided Widenings

When widening a bridge deck on one side, the opposite side shall also be assessed for barrier adequacy described herein. In more cases, when bridge is widened on one side, the barrier on the opposite side is replaced so that both barriers have aligned service life and consistent aesthetics.

Heritage and Aesthetic Considerations

Heritage structures and aesthetic requirements, where applicable, shall be evaluated on a case-by-case basis in consultation with the MTO Structures Office.

Situations Where Barrier Upgrades May Not Improve Safety

Barrier upgrades shall not be undertaken if they would reduce safety, such as by constraining sight lines, narrowing shoulders, or reducing lane widths, unless these broader operational limitations are addressed as part of the project.

P2-2.1.5 Sidewalks, Curbs and Medians

Minimum Curb Height for Sidewalks & Handrail Review

Where deck rehabilitation results in a sidewalk curb height less than 150 mm, or where the existing curb height is already less than 150 mm, the sidewalk shall be refaced to provide a minimum 150 mm curb, or other pedestrian protection shall be provided.

When sidewalks are refaced, handrail heights shall be reviewed and adjusted as required.

Exception for Medians and Parapet Walls

A curb height less than 150 mm may be acceptable for:

- medians, and
- curbs adjacent to concrete parapet walls, provided the parapet walls meet *CSA S6 CHBDC* minimum height requirements.

Metal Lattice and Open Railing Systems

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Rehabilitation shall not reduce curb height below 150 mm where metal lattice or open railing systems are present. Where reduction is unavoidable:

- the curb shall be refaced to restore 150 mm height, and
- the railing shall be raised, unless the railing face is flush with the curb edge, in which case raising may not be required.

All modified railing systems shall conform to *CSA S6 CHBDC*.

Median Removal

Medians may be removed during rehabilitation to facilitate lane shifts, as required.

P2-2.1.6 Approach Slabs

When the existing approach slab needs to be partially removed to access and complete repairs to adjacent components such as the ballast wall, wingwalls etc., complete replacement of the approach slab is preferred, as it is often difficult to tie-in to the remaining approach slab due to settlement.

P2-2.2 Substructure Components

A flowchart and decision-matrix table were included in the 2007 Structure Rehabilitation Manual (Appendix D Figure 2.D-1 and corresponding Table 2.D-1) for rehabilitating substructure components. The original flowchart and table have been reorganized in Figure 11.

The criteria are not meant to be rigid but are intended as a guide. The chart is more applicable to abutment and pier walls but suitable for pier columns too. However, the rehabilitation options for slender pier columns are more limited because extensive removals could result in expensive staging and requiring temporary support systems. A similar treatment would also be applicable to circular columns where it is difficult to remove concrete behind the tightly spaced spiral reinforcing steel.

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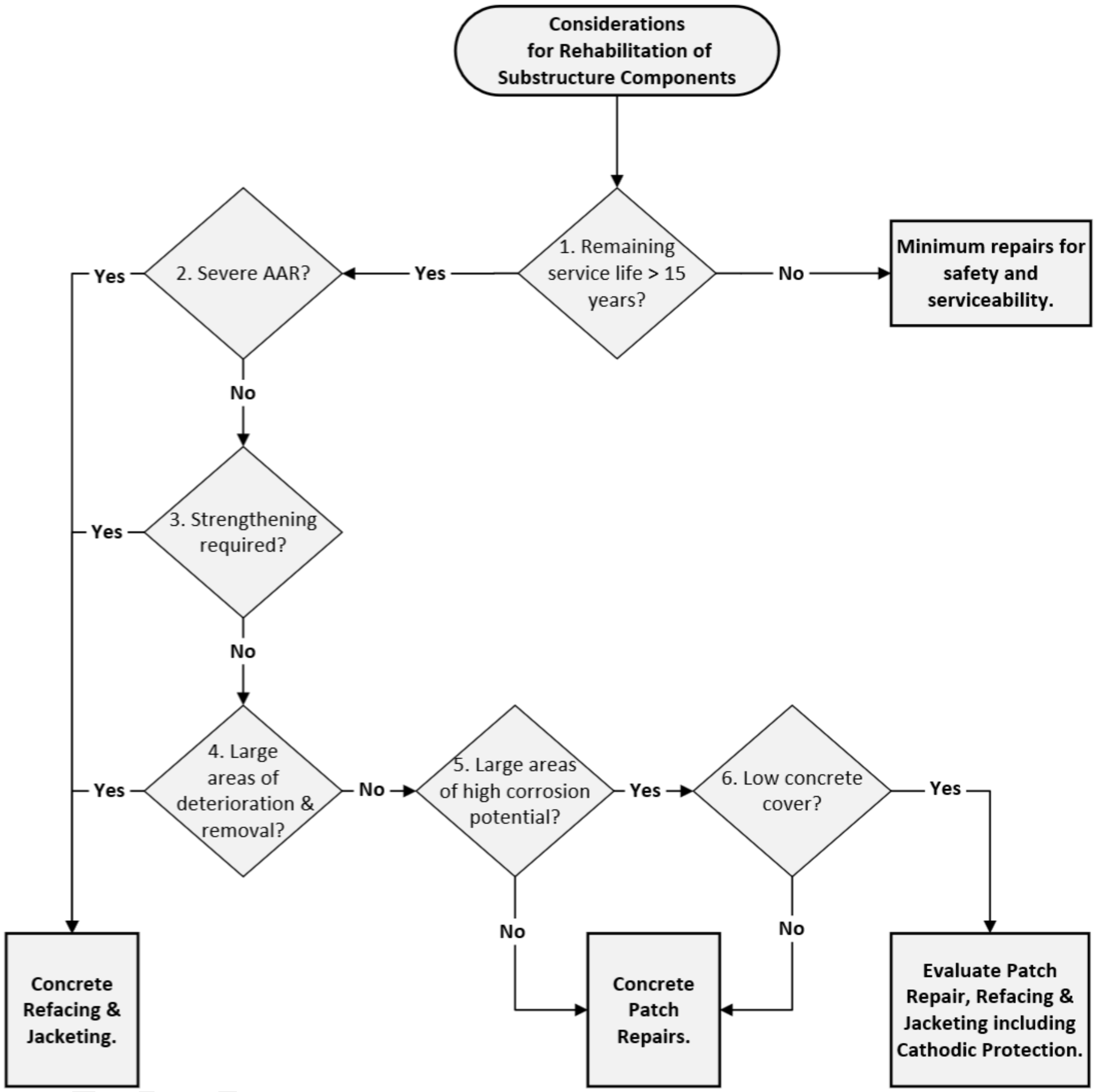
REHABILITATION SELECTION



P2-3.4 Refacing and Jacketing

P2-3.2 Concrete Patch Repairs

P2-3.2 Concrete Patch Repairs
P2-3.4 Refacing and Jacketing
P2-3.8 Cathodic Protection

Figure 11 – Considerations for Selecting Rehabilitation Strategy for Substructure Components.

REHABILITATION SELECTION

When the bridge is **expected to be replaced within 15 years (◇1**, *corresponds to the numbered decision points in the flowchart***)**, work shall be minimized and focused on maintaining the safety and operation of the bridge.

Alkali-aggregate reactions (◇2) are expansive and cause the concrete to crack. AAR reactions are irreversible, the rate of reaction is influenced by the ingress of moisture. Rehabilitation strategies are generally aimed at slowing the rate of moisture ingress by increasing concrete cover, such as refacing or jacketing. However, because AAR-related deterioration continues, despite mitigation, reconstruction or replacement is eventually warranted.

In the 2007 Structure Rehabilitation Manual (Figure 2.D-1), **large areas of expected removal (◇4), or high corrosion potential (◇5)** were generally characterized as affecting more than 30 percent of a component's area, and **low concrete cover (◇6)** was identified as less than 20 mm. These values were not intended as strict thresholds. Instead, they should be used as indicators to guide the selection of an appropriate rehabilitation strategy, particularly when determining whether localized patch repairs or uniform refacing of the entire surface is warranted.

The extent, distribution, and severity of deterioration are critical factors in selecting a repair method. Refacing an entire component when deterioration is limited to a few small, isolated areas is typically unnecessary and not cost-effective. However, when deterioration is widespread and dispersed throughout a component, the cumulative quantity of repairs may increase to the point where the unit costs of refacing and patching become comparable. Engineers should evaluate these factors collectively to determine the most efficient and durable treatment approach.

Large areas of high corrosion potential (◇5) and/or **low concrete cover (◇6) are** direct indicators of elevated corrosion risk. Corrosion potential reflects the concrete's susceptibility to moisture and chemical penetration, while the concrete cover represents the degree of protection provided to the embedded reinforcing bars against external factors. Both aspects must be evaluated together with the structure's remaining service life to determine the most appropriate rehabilitation strategy.

When the existing deterioration, environmental exposure condition, and the structure's remaining service life are consistent with a localized repair strategy, a targeted approach, patching only the areas requiring intervention, is appropriate. However, when these factors indicate widespread or progressing deterioration, or when the underlying cause of damage remains active, a more comprehensive rehabilitation strategy may be required. This could include uniform refacing of the affected surface, addressing the source of contamination (e.g., repairing a leaking expansion joint), or implementing both measures to ensure long-term performance.

P2-2.2.1 Ballast Walls

- Accessible areas of ballast walls may be repaired using concrete patching.
- Severely deteriorated ballast walls shall be reconstructed.
- Approach slab replacement and backfill excavation may be required to allow ballast wall removal and reconstruction.
- Replacing a ballast wall in poor condition provides improved access to repair the ends of thick deck slab soffits and concrete girders.
- Where deterioration is caused by pressure from adjacent concrete pavement, relief joints shall be installed in the approach pavement to prevent recurrence.

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- Relief joints should also be installed wherever concrete pavement abuts a ballast wall.

P2-2.2.2 Wingwalls

This section replaces Bridge Office Design Bulletin dated May 4, 2011.

Prior to the implementation of the wingwall design aids in the 2008 update of the *Structural Manual* (Revision #55), wingwalls were designed primarily for earth pressure without consideration of any traffic impact load that could be imposed on the traffic barrier supported on the wingwall. The design aids implemented in 2008 considered traffic impact load to be resisted by the wingwall in conjunction with the earth pressure; consequently, the main flexural reinforcement in the wall was increased substantially compared with previous designs. There is, however, no evidence of failure of any existing wingwalls due to traffic impact, and any strengthening for such effect would be costly and may not be justified.

Existing wingwalls that are adequate for earth pressure and do not show any signs of flexural distress do not have to be strengthened for additional horizontal bending due to traffic impact load.

If part of the ballast wall is removed for semi-integral abutment conversion, the structural adequacy of the remaining wingwall for earth pressure must be evaluated and strengthened accordingly. However, traffic impact effect does not have to be included in the evaluation and strengthening.

When an existing traffic railing on wingwall is to be replaced with new barrier/parapet wall, particular attention must be given to the transfer of traffic impact load from the barrier system to the wingwall. The dowels for the new barrier/parapet wall may have to be lengthened for proper tension lap with the vertical reinforcement in the wingwall.

P2-2.3 Buried Structures

P2-2.3.1 Reinforced Concrete Culverts

Results from a study of minor rehabilitations on over 60 culverts showed that structural costs accounted for an average of 30% of total construction costs for standalone culvert rehabilitation projects. Temporary works, access, and traffic staging represent significant fixed costs that must be considered in the programming and treatment strategy selection for the work.

Rehabilitation of reinforced concrete culverts should generally be deferred unless significant performance issues need to be addressed. Rehabilitation is typically not required at intervals applied to asset management of bridges. Reinforced culverts are robust, have little to no replaceable components and typically do not require intervention for 50 years or longer depending on the performance and level of deterioration. Minor localized concrete deterioration and seepage generally do not warrant intervention.

P2-2.3.1.1 Crack Repairs in Concrete Culverts

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Unless the culvert is used by pedestrians or recreational vehicles, minor seepage does not warrant repair. If seepage is a symptom of deformation, undermining, loss of backfill or similar structural issues, the underlying cause must be addressed, as repairing the cracks alone will be ineffective.

A recent Ministry review of over 70 culverts with cracks repaired using crack injection found that crack injection with polyurethane is largely ineffective for completely cutting off water seepage into culverts. Results showed that approximately 15% of the injected cracks began leaking within two years of repair, nearly 50% exhibited leakage within four years, and upwards of 70% were leaking within 10 years. Therefore, crack injection with polyurethane, according to *OPSS.PROV 932*, shall not be specified for sealing cracks in culverts for the purpose of seepage control. Crack injection repairs with epoxy outlined in *OPSS.PROV 932* and described in Section P2-3.6 Concrete Crack Repairs, may be specified where critical restoration of structural capacity is required. Specific treatment of cracks below the water table shall be accounted for in design.

P2-2.3.1.2 Patch Repairs in Concrete Culverts

For concrete culverts, localized concrete repairs and small quantities of repair results in high relative costs and should be avoided. Designers shall consider the impact of minor localized deterioration on structural performance and serviceability. Concrete patch repairs, when required, shall be specified as described in Section P2-3.2 Concrete Patch Repairs, however deferring repair shall be considered, where practical, and lining or replacement should be considered in preference to localized patch repairs once warranted based on the overall barrel condition.

P2-2.3.2 Soil-Metal Culverts

Soil-metal culverts derive their structural capacity and stability from its interaction with the surrounding engineered fill. Common forms of deterioration include perforation, corrosion, deformation of the steel plates, and separation along the bolt lines. Isolated perforation may be repaired locally; however, this approach becomes ineffective where perforations are large, deformed or widespread. Significant perforations may indicate loss or undermining of the surrounding fill, potentially compromising the soil support. Consequently, repair measures for soil-metal culverts are generally aimed at arresting further deterioration, rather than restoring the culvert to its original structural shape or capacity.

Installation of internal struts to limit ongoing deformation are considered a short-term emergency repair only. Full circumferential repairs are generally required where severe deformation is observed.

Invert paving has been used to address corrosion along the base; however full lining systems may offer improved durability and extended service life for relatively little additional cost. Repair strategies for soil-metal culverts should prioritize long-term structural performance and lifecycle value over short-term defect correction. Section P2-3.16 Liners for Structural Culverts discusses options, considerations and challenges related to lining soil-metal culverts.

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P2-3 Rehabilitation Treatments

P2-3.1 Concrete Removal

P2-3.1.1 Background

Rehabilitation of reinforced concrete structures generally involves removing deteriorated concrete, preparing the remaining surfaces, and placing new concrete. Removal areas and quantities should be determined based on available information, including inspection reports, detailed condition surveys, and site visits. This work often requires coordination with other functions and may also require approval from external agencies, particularly when removals occur over watercourses, rail corridors, or other regulated areas. Early engagement with affected stakeholders is essential to identify and address their requirements, such as constraints on duration, noise, debris containment, and allowable work windows to minimize the risk of project delays. The type, location and extent of concrete removal influence the method of rehabilitation.

Chloride ingress is the primary cause of corrosion of embedded reinforcing steel within concrete. Historically, the Ministry addressed this risk by removing chloride-contaminated concrete during rehabilitation, based on half-cell corrosion potential readings. Over time, the requirements for removing chloride-contaminated concrete based on corrosion potential measurements have been revised several times.

P2-3.1.2 Construction Details

P2-3.1.2.1 Types of Concrete Removal

Types of concrete removal are differentiated based on depth and location. Full depth involves removing the entire thickness of decks, barrier walls, approach slabs etc., either partially or along the entire length. Full depth removals often have an impact on the structural adequacy, stability, and integrity of a structure, which must be considered and addressed during design.

Partial depth removals are categorized in OPS Specifications as Type A, B and C, based on their location. Type A refers to partial depth removals from top surfaces, Type B for removals from overhead surfaces typically on soffits and fascia, and Type C for removals from surfaces not covered in Types A and B. The type of partial depth removal influences the method of patch repair, refer to Section P2-3.2 Concrete Patch Repairs.

P2-3.1.2.2 Equipment

Equipment requirements are detailed in *OPSS.PROV 928* and briefly described below. Common removal equipment, such as air hammers, breakers, scarifiers, operate mechanically by striking the concrete surface. These methods impact the remaining surface, causing micro-fractures in the substrate. Additionally, small vibrations are transmitted through reinforcing bars, causing additional cracks in the remaining concrete. The equipment limitations specified in *OPSS.PROV 928* are developed to reduce these adverse effects.

Chipping hammers are the lightest pneumatic air hammers, weighing up to 9 kg, and with a maximum 102 mm piston stroke. They are permitted in all removal areas, unless otherwise

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specified, and only chipping hammers can be used for partial-depth concrete removals. Chipping hammers have a low production rate but can be used to remove difficult to access areas.

In 2014, the Ministry completed a trial using a Position Actuator Manipulator (PAM) where a chipping hammer was mounted on a hydraulic boom carrier. The trial divided an abutment wall into three sections and removed concrete using three methods:

- PAM only.
- PAM initially and chipping hammer for the last 25 mm.
- Chipping hammer only.

Upon completion, the sections were visually inspected, sounded, and tested. The section removed using chipping hammer had the least number and narrowest surface cracks. By comparison, the section removed using only PAM had the most and widest surface cracks, the most amount of surface damage in the remaining concrete and lowest tensile bond strength. PAM or similar equipment are not permitted for partial depth repairs. Where the contractor proposes to use removal equipment not covered in *OPSS.PROV 928*, the Regional Structural Office must be consulted.

Jack hammers can weigh up to 14 kg. They are more powerful than chipping hammers. They are not permitted for partial depth removals, within 100 mm from concrete to remain in place, within 25 mm of any reinforcing steel to remain in place, or within 100 mm from the edges and faces of structural steel members to remain in place.

Rig-mounted breakers and concrete crushers are powerful concrete breaking machines typically used for demolition. They are not permitted to be used in the vicinity of any concrete or reinforcing steel to remain because these machines are less precise and there is a higher risk to damaging surrounding components.

Scarifying equipment are equipped with powerful cutting teeth attached to a rotating drum. They are used to remove concrete from a horizontal surface, typically the deck top, to a specified depth. In some cases, a scarifying attachment may be used to uniformly remove concrete, within the concrete cover, on vertical surfaces. Equipment used for scarifying must be capable of removing a minimum of 6 mm of concrete from the top surface of the structure in one pass. Deeper removals require multiple passes which increases the risk of microcracking.

Hydro-demolition equipment uses high-pressure water to remove concrete by breaking it into smaller pieces. The high-pressured water does not induce significant vibration, thus reducing microcracking in the remaining concrete. It can be used to remove concrete partial depth without damaging the reinforcing bars. It also produces a course profile which provides good bond between the repair and substrate materials. The depth of removal is controlled by adjusting the water pressure. Where the concrete is delaminated or weaker, the depth will increase for a given pressure and there is a higher risk of full depth ‘blow out’.

A key consideration for hydro-demolition is the control and collection of effluents. As the water breaks the concrete, it mixes with fine particles from the broken concrete to form a ‘sludge’ which is then collected. This method is not suitable for small, isolated patches and more appropriate for removing large areas.

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P2-3.1.2.3 Preparation of Concrete Surfaces and Reinforcing Steel

A proper bond between new concrete and the existing substrate is critical to the durability of the repair. To achieve this, the existing concrete surfaces must be cleaned and roughened, the reinforcing steel bars to remain must be cleaned, and the concrete surface must be sufficiently wetted prior to placing new concrete. These requirements are described in detail in *OPSS.PROV 904* and *929*.

Existing epoxy-coated reinforcing steel does not require the epoxy coating to be cleaned off in areas of repair, however a light blast clean is recommended to remove any loose coating and debris.

P2-3.1.3 Design Requirements

P2-3.1.3.1 Concrete Removal Policy

Concrete removal shall be by delaminated concrete identified by sounding. In addition, deteriorated concrete may be removed at the designer's discretion, including areas of severe scaling, and areas of extensive cracking. Light honeycombing, abrasion, and shallow pop-outs typically do not warrant repair. Engineering judgement shall be applied to determine the extent of repair required. Areas of deteriorated concrete identified in the condition assessment as requiring repair shall be indicated in the contract documents via scaled detail drawings clearly delineating areas to be removed. The designer shall not identify areas for repair that do not show evidence of deterioration. The Contract Administrator will confirm the removals limits in the field during construction for the elements noted in the contract documents.

Removal of sound concrete by corrosion potential is not permitted unless approved by the Head of the Structural Section.

The designer shall clearly define the concrete removal sequence in the contract documents, including any removals permitted outside the active construction stages. The designer shall also identify all applicable restrictions, such as sub-staging requirements, that apply to all concrete removal activities.

Rationale (Informative)

Historically, the Ministry's approach to managing chloride-induced corrosion involved removing chloride-contaminated concrete based on half-cell corrosion potential readings. This practice was formalized in 1989 and later refined in 2004. The intent was to extend service life by eliminating concrete most at risk of reinforcing steel corrosion.

However, a recent review of condition survey data and past rehabilitation projects revealed that this approach was rarely implemented in practice. While half-cell measurements remain useful as a relative indicator of corrosion activity, they are highly sensitive to environmental factors such as humidity and temperature, making accurate estimation of removal quantities difficult. Administering this work during construction proved challenging, and most waterproofed decks simply did not meet the 2004 criteria for chloride content and corrosion potential.

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The cost and uncertainty of removing sound chloride-contaminated concrete outweighed the benefits. Common mitigation strategies, such as replacing waterproofing, overlays or cathodic protection, do not remove chlorides but they can slow further ingress and mitigate their impact.

The most effective strategy remains prevention, by stopping chlorides from entering the concrete. Once chlorides are present, managing the associated risks through protective measures is far more practical and cost-effective than attempting removal. For these reasons, removing concrete based on corrosion potential is no longer part of the Ministry's standard procedure.

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P2-3.1.3.2 Avoid Excessive Removal

Excessive removal of concrete or removing concrete in the wrong sequence can seriously affect the capacity, stability, and behavior of both a component and the bridge.

Concrete is typically removed to a depth of 25 mm behind the first layer of reinforcing bars where the concrete is sound. In areas of unsound concrete, the depth of removal may extend further until sound concrete is reached. This may result in excessive concrete removals and disengage reinforcing bars over large areas of a component. The sequence of removals and need for temporary support systems must be carefully assessed and addressed during design, especially when significant removals, deep removals, or removals in critical areas are involved.

The designer must consider the location and extent of removals required for each component involved, and its effect on the entire bridge. Measures must be taken to ensure adequate strength and stability during removals, such as placing the component on temporary supports or staging the removals.

Where excessive removals would adversely affect the capacity of the component and bridge or result in expensive or impractical staging and temporary support systems, the designer should consider alternative methods of rehabilitation that do not involve such removals.

Concrete must not be removed such that the main reinforcing steels are no longer adequately anchored, embedded, or surrounded by sufficient concrete to transfer loads to the reinforcing steel, or develop the strength of the reinforcing steel. Concrete must not be removed under any circumstances to the point where the capacity of the component and the structure is reduced to less than adequate to support the applied loads unless adequate mitigating measures (such as temporary supports) are provided.

Below are examples of areas where mitigating measures may be required to avoid excessive removals, including:

- Extensive removal of concrete in slender compression components, such as around the circumference and through the depth of pier columns and shafts.
- Extensive removal of concrete in the compression zones of reinforced and prestressed concrete (girders, T-beams, and slab) bridges.
- Extensive removal of concrete directly over bearings and supports, and in concrete bearing seats.
- Extensive removal of concrete which exposes prestressing steel or ducts, particularly at anchorages and hold-down locations.

- Extensive removal of concrete which exposes main tension reinforcing steel over a significant length, in the following areas:
  - At anchorage zones and over its development length.
  - Around the negative moment region in rigid frame bridges (slab and T-beam).
  - Over the top of concrete girders in negative moment regions at supports in continuous cast-in-place reinforced concrete bridges.
  - In the positive moment regions in concrete beams and slabs.

P2-3.1.4 OPS Specifications and References

Relevant OPS specifications, non-standard special provisions, and contract design, estimating and documentation (CDED) chapters related to concrete removal are summarized in Table 20.

Table 20 – OPS Specifications and References Related to Concrete Removal.

Means and Methods	OPS Specifications	NSSP (Custodian Offices)	CDED Chapter
Concrete Removal	OPSS.PROV 928	-	-
Surface Preparation Requirements	OPSS.PROV 904, 930	-	-
Abrasive Blast Clean	OPSS.PROV 929	-	-
Hydro-Demolition	-	Yes (Structures Office)	-

A comprehensive repository of MTO technical documents can be found on the MTO Technical Publications website.

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P2-3.2 Concrete Patch Repairs

P2-3.2.1 Background

Concrete patch repair involves removing localized areas of deteriorated concrete and reinstating them with an appropriate repair material, typically normal concrete. This method is used when the extent and distribution of deterioration do not justify more extensive measures, such as overlays, refacing, or full component replacement.

P2-3.2.2 Construction Details

P2-3.2.2.1 Patch Repair using Normal Concrete

Normal concrete, as defined in the Structural Manual, is suitable for patching vertical, horizontal, and overhead surfaces and is appropriate for all concrete components where there is sufficient access to place the concrete.

For patches where reinforcing bars are exposed, the removal depth should be minimum 100 mm, with 25 mm clearance locally beneath the first layer of reinforcement. Engineering judgement is required where reinforcing steel is not exposed; shallow patches may be used but they should be reinforced with wire mesh or additional sound concrete removal may be required in order to be anchored with existing reinforcing bars. Shallow patches are not suitable for deck tops; those patches should be anchored with the existing reinforcing bars to ensure adequate bond under traffic loading.

Normal concrete is readily available and economical to place, however the distance to a certified ready-mix plant should be confirmed in design to ensure discharge requirements can be met. Normal concrete is most compatible with existing concrete components and less prone to shrinkage cracking compared to concrete with silica fume and proprietary products. Normal concrete is relatively permeable to water and chloride ions. If the source of chloride is not eliminated, corrosion of reinforcing steel will reinitiate over time as chlorides migrate through the newly patched concrete to the reinforcing steel. However, with adequate concrete cover, patches are reasonably durable in areas exposed to chloride ions.

Concrete is directly poured into horizontal patches and vertical patches that are accessible from the top. Concrete is placed using 'form and pump' for overhead patches where concrete is pumped directly into an enclosed formwork through injection ports. Vertical clearance requirements must be considered for overhead repairs as formwork and supporting hangers extend below the surface. Form and pump can be used to place concrete in vertical patches where the patched area needs to remain flush with surrounding surface.

In general, overbuilding patches can simplify concrete placement - for example, allowing vertical patches to be poured directly from above - and may be acceptable in low-visibility areas where aesthetics is not a concern. However, overbuilt patches on the soffit, fascia, or concrete girders can create protrusions and visually undesirable patterns. Although overbuilding increases concrete cover, its benefit is limited when the entire surface has insufficient cover, as only the patched areas gain additional protection. A more effective alternative is to build the patch flush and followed by wrapping with FRP (refer to Section P2-3.12 Repairs and Strengthening using

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Fibre Reinforced Polymers (FRP)), which enhances durability and provides a uniform surface that blends better with the surrounding concrete.

P2-3.2.2.2 Patch Repair using Shotcrete

This section replaces Bridge Office Memorandum #2014-02, dated May 15, 2014.

Shotcrete is a mixture of aggregates, cement and water that is projected onto a surface using high velocity air pressure. ACI 506-16 Guide to Shotcrete provides comprehensive information on shotcrete.

Supplementary cementing materials are often added to shotcrete mix, with silica fume being the most common. Silica fume in shotcrete increases strength, decreases permeability, and enhances cohesion and adhesion. While prebagged normal shotcrete (without silica fume) is available, it offers little cost savings compared to silica fume shotcrete, and its use is not recommended except in cases where a cathodic protection system exists, and electrical conductivity is a consideration.

The quality of a shotcrete repair is highly dependent on the expertise of the nozzleman. The Ministry's Engineering Materials Office, Concrete Section has a certification program and maintains a list of qualified personnels.

Due to the very fine nature and high reactivity of silica fume, wet curing is essential for silica fume shotcrete to ensure bond strength development and reduce the risk of cracking. Curing silica fume shotcrete requires an initial one-day continuous fog mist curing followed by three days of continuous fog mist or wet burlap curing. Managing traffic and gaining access to the shotcreted area during the four days of continuous curing can often be challenging. Shotcrete repairs may be considered for holding strategy applications where reduced durability is acceptable. In such cases, if the specified curing cannot be met, a reduced curing regime may be considered. It must consist of minimum 4 to 6 hours of fog mist immediately after placement, followed by two coats of curing compound. However, the implications of reduced curing on quality and durability must be recognized.

Shotcrete is appropriate for large and shallow repair areas especially where formwork is impractical or not feasible. It is suitable for both vertical and overhead applications, including emergency and temporary repairs. For small quantities, other methods are more cost-effective. Shotcrete is typically placed in 25 to 50 mm thick layers to prevent sagging. To minimize shrinkage cracking, a galvanized or stainless-steel wire mesh is anchored into the concrete or fastened to the exposed reinforcing steel within the repair area. It is not advisable to shotcrete areas thicker than 200 mm, areas with closely spaced reinforcing bars (less than 75 mm clear spacing) or multiple mats of exposed reinforcing bars as the shotcrete may not fully penetrate those areas. Shotcrete cannot be properly applied where there is insufficient room to position the nozzle at right angle to the surface. Finally, shotcrete should not be used for components in prominent visible locations if aesthetics is important.

P2-3.2.2.3 Patch Repair using Proprietary Patching Materials

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Manufacturers have developed proprietary products for a variety of applications. The basic ingredients of proprietary products are cement, sand and special additives or modifiers to enhance specific properties. Different products have different formulations, including with cementitious, non-cementitious, polymer, or magnesium phosphate binders. Concrete Section maintains a list of approved proprietary materials and the list is updated on an ongoing basis.

OPS Specifications permit contractors to propose and use proprietary patching materials to repair patches where the longest dimension is less than 400 mm. This application is covered under *OPSS.PROV 930* and included as part of the '0930-XXXX' concrete patch repair items. It does not require a separate tender item in the contract.

When it is known during design that patching with normal concrete is not feasible and proprietary patching products must be used, it shall be specified using the appropriate non-standard tender item (NSTI). A flowchart is provided in Section P2-3.2.4.1 Selecting Concrete Patching Materials to guide patching material selection based on repair size, curing time requirements, and service condition. Engineering Materials Office, Concrete Section maintains and periodically updates a list of approved proprietary patching products.

There are several considerations regarding the use of proprietary materials, including:

Application – Proprietary products are formulated to enhance certain properties and are not always suitable for all applications. Some applications and enhanced properties include:

- **Trowel-applied:** These products can be applied to vertical and overhead surfaces without the use of formwork.
- **Self-levelling:** These products flow readily and are suitable for horizontal surfaces in difficult-to-reach areas.
- **Rapid hardening:** These products can reach 20+ MPa in 10 to 24 hours.
- **Rapid setting:** These products can reach 20+ MPa in 4 hours or less.

Yield and Working Time – Proprietary products are commonly packaged in 55 lb (25 kg) bags and yields about 0.01 m^3 per bag. Some have working time less than 15 minutes. It is labour intensive and challenging to place in large quantities.

Aggregates and Patch Thickness – For deep repairs, care must be taken to prevent concrete temperatures from exceeding 70°C and to limit the temperature gradient within the same section. Proper temperature control helps to prevent potential long-term degradation due to delayed ettringite formation (DEF) and thermal cracking. The high internal hydration temperatures of some mixes and the exothermic reactions of some polymer systems pose challenges as the thickness of the repair increases.

Furthermore, proprietary products without aggregates are essentially mortar rather than concrete. They can be used directly for thin repairs but will experience high rates of shrinkage when placed in thicker sections. Requirements for individual products vary widely and manufacturer's literature must be consulted to determine if the products can be extended with the addition of aggregates for repairing deeper patches. Those that can be extended generally have specific requirements

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for the aggregate type and gradation. Using proprietary products for deep repairs is generally discouraged.

Ambient Temperature – For many proprietary products, their setting and curing times, as well as the rate of strength gain, are much slower in cool temperatures than during warm weather. The application of heat to increase substrate or ambient air temperature may be practical in some patching applications, but not in others. It is important to consider temperature effects and to schedule rapid concrete repairs work during warmer months or provide for the maintenance of warm temperatures in some other way to increase the likelihood of a successful rapid repair.

Abrasive resistance – Some products have low abrasive resistance and are not suitable for repairing surfaces exposed to traffic.

P2-3.2.3 Design Requirements

P2-3.2.3.1 Electrochemical Compatibility

Corrosion is a multifaceted and highly complex process. At the project level, it is neither realistic to determine the relative contribution of each corrosion mechanism for every individual patch, nor practical to develop a customized repair mix for each location based on the specific physical or chemical properties of the substrate. Nonetheless, a general understanding of the governing corrosion mechanisms is essential. It helps set realistic expectations for the performance of patch repairs.

Corrosion of reinforcing steel in concrete is an electrochemical process, idealized in Figure 12.

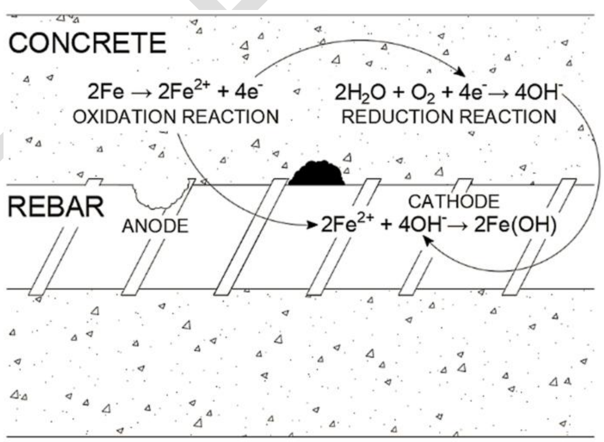

Figure 12 – Idealized electrochemical process for corrosion of reinforcing steel in concrete, modified from [8].

Steel (iron, Fe) undergoes oxidation at anodic regions of the reinforcement, where iron atoms release electrons and dissolve as ferrous ions (Fe^{2+}). The released electrons travel through the steel to cathodic regions, where they are consumed in a reduction reaction involving oxygen and water, producing hydroxide ions (OH^-). The ferrous ions generated at the anode subsequently react with hydroxide ions and oxygen, forming iron hydroxides and hydrated iron oxides (rust), which accumulate primarily at or near the anodic locations.

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Rust occupies larger volume than the original steel, it exerts pressure on the surrounding concrete, which can lead to cracking, spalling, and delamination. Corrosion occurs at the anode, while steel at the cathode remains protected. Cathodic protection systems use this principle by connecting the steel requiring protection to a more reactive sacrificial metal that acts as the anode (refer to Section P2-3.8 Cathodic Protection). In reinforced concrete, the anode and cathode can be within the same bar or different bars that are tied together.

Following a patch repair, corrosion may continue either within the repaired area or surrounding concrete through both microcell and macrocell corrosion mechanisms. Microcell corrosion occurs when the anodic and cathodic reactions take place in very close proximity, often within the same localized area of steel. Macrocell corrosion, by contrast, occurs when these reactions develop at visibly separate locations, either along different parts of the same bar or between connected bars [8]. Macrocell corrosion is sometimes described as 'ring-anode' or 'halo' effect. Both mechanisms can occur independently or at the same time.

Before repair, the damaged area typically acts as the anode because it is more corrosive to the steel than the surrounding substrate; as a result, the steel in the adjacent substrate is relatively cathodically protected. After the chloride-contaminated concrete is removed and replaced with low-chloride repair material, the surrounding concrete often contains higher chloride levels than the patched area. This can reverse the electrochemical conditions: the steel in the substrate may lose its previous cathodic protection, become anodic relative to the repaired zone, and begin to corrode [9].

P2-3.2.3.2 Repairing GFRP-Reinforced Barrier Walls

Since 2005, the Ministry begin using GFRP bars in bridges, especially in barrier walls.

Some damages to the embedded reinforcing bars are unavoidable when removing concrete around them. FRP bars are not fragile, but they are brittle and less ductile compared to steel bars. With its high ductility, when a steel bar is damaged by a chipping hammer, it undergoes plastic deformation locally around the damaged area when subjected to stress. This plastic deformation allows steel bars to redistribute the applied stress, effectively transferring the load to adjacent parts of the bar. As a result, even after a steel bar is damaged, it can retain its ability to carry the load, albeit at a reduced capacity. In comparison, when FRP bars are damaged, due to their limited ductility, they do not effectively redistribute stress around the damaged area, and this leads to a more abrupt loss of load carrying capacity. Further, GFRP is a composite material of glass fibres in a resin matrix. FRP are also anisotropic and rely on fibres for their tensile-carrying capacity; damage that severs fibres directly reduces the load-carrying capacity of the reinforcement. When the fibres are cut, it will have a disproportional impact on the load carrying capacity of GFRP bars compared to a steel bar with a similar reduction in cross-section.

Another challenge with removing concrete around FRP bars is to completely remove the concrete paste from the bars. Remaining paste on the bars can act as a bond breaker between the GFRP bars and new concrete. When pouring concrete for new GFRP-reinforced barrier walls, many contractors have adopted the practice of protecting the GFRP bars with plastic sleeves during the initial concrete pour up to the base of the barrier walls to avoid concrete splattering onto the bars.

The Ministry currently specifies a threshold for bar damage in *OPSS.PROV 950* where:

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1. Any damage resulting in visible fibres, or any cut or defect greater than 0.7 mm deep for bars of size G15 or less, and 1.0 mm deep for larger bars is not acceptable.
2. All visible damage to the bars exceeding 2% of the surface area per 300 mm length of bar shall be repaired by lap splice.

These thresholds are intended for new construction and are not considered practically achievable when removing cured concrete around FRP bars with conventional means such as using pneumatic hammers or hydro-demolition. As a result, retained FRP bars after the removal of concrete are not relied upon for resisting loads as per *CSA S6-25 CHBDC* Clause 16.11. Repairs generally involve saw-cutting removals and doweling in new stainless-steel bars. The Ministry currently does not have provisions for FRP dowels.

P2-3.2.4 OPS Specifications and References

Relevant OPS construction specifications, non-standard special provisions (NSSP) and contract design, estimating and documentation (CDED) chapters related to concrete patch repairs are summarized in Table 21.

Table 21 – OPS Specifications and References Related to Concrete Patch Repairs.

Means and Methods	OPS Specifications	NSSP (Custodian Offices)	CDED Chapter
Normal Concrete	OPSS.PROV 930	-	B930
High-Early Strength Concrete	-	Yes (Concrete Section) (1)	
Rapid Hardening Concrete	-	Yes (Concrete Section) (1)	
Silica Fume Shotcrete	OPSS.PROV 931	-	
Proprietary Patching Products	-	Yes (Concrete Section) (1)	

(1) *Requirements will vary depending on application and project specific constraints. Consult with Concrete Section and Structures Office when high-early strength, rapid hardening concrete and proprietary patching products are required for patches or applications larger than 400 x 400 mm.*

A comprehensive repository of MTO technical documents can be found on the MTO Technical Publications website.

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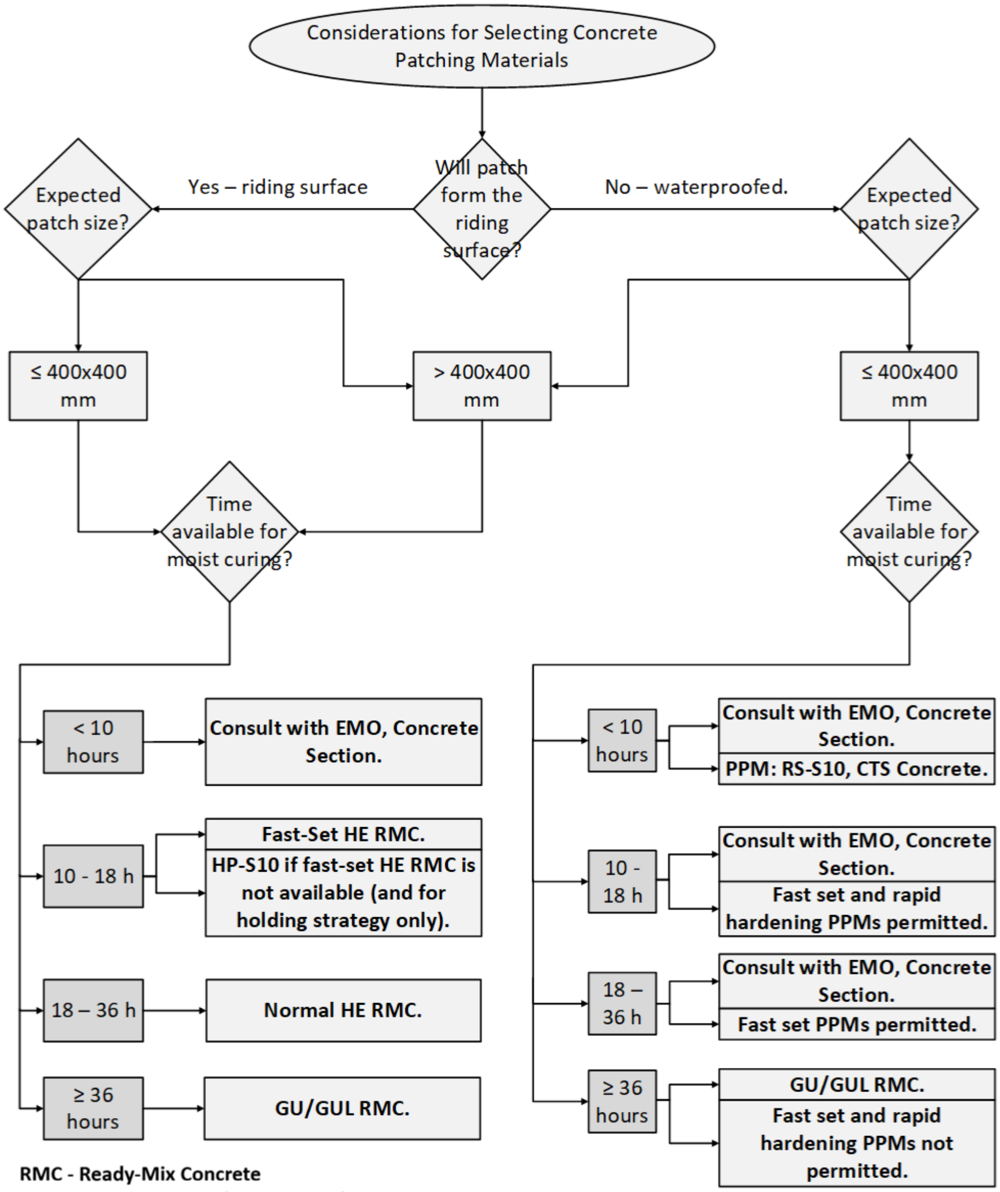
P2-3.2.4.1 Selecting Concrete Patching Materials



Note: Proprietary patching materials shall be commercially manufactured concrete repair products specifically intended for the proposed application. All patching materials shall be compatible with the existing concrete substrate, exposure conditions, and service environment. Magnesium phosphate–based materials shall not be permitted in locations that are adjacent to, in contact with, or within the vicinity of reinforcing steel.

All ready-mixed concrete used to form the final riding surface shall comply with OPSS 1002, including all applicable requirements of LS 601 and LS 607. The finished riding surface shall receive surface grooving in accordance with the Contract Documents to achieve the specified friction and skid resistance performance.

P2-3.3 Overlays

P2-3.3.1 Background

This method involves placing a layer of new concrete on a properly prepared concrete deck. The most widely used material for MTO bridge deck overlays is 30 MPa normal concrete. The Ministry has used latex modified concrete and silica fume concrete overlays, polyester polymer concrete and ultra-high-performance concrete fibre reinforced (UHPFRC) overlays.

Concrete overlays provide additional cover to reinforcing steel and are well suited to repair extensively spalled and scaled decks. The rate of corrosion and corrosion damage are slowed down due to the increased concrete cover and possible upward migration of chloride from the original concrete into the overlay. However, if the chloride content at the rebar level is very high (> 2x threshold), this method may not stop active corrosion if the chloride contaminated concrete is not removed in areas indicating corrosion activity. Wide cracks in existing concrete would likely be reflected in the concrete overlays.

P2-3.3.2 Construction Details

P2-3.3.2.1 Normal Concrete Overlay

30 MPa normal concrete overlays are by far the most common type of overlay on MTO bridges. The minimum thickness for a normal concrete overlay is 45 mm, as specified in *OPSS.PROV 930*. This minimum thickness requirement is related to maximum aggregate size; it ensures aggregates flow freely during placement and distribute evenly throughout the placement area.

In general, the specified thickness of a normal concrete overlay on MTO projects is 60 mm from the scarified surface. This accounts for unevenness in the existing concrete substrate and allows for minor grade corrections while still meeting minimum thickness requirements.

OPSS.PROV 1002 addresses material requirements for aggregates, specifying 13.2 mm nominal maximum aggregate size for repairs less than 100 mm thick and a 19 mm nominal maximum aggregate size for repairs exceeding 100 mm thickness. Where an overlay is specified alongside deck cantilever reconstruction, the varying thicknesses may necessitate two mix designs - one using 19 mm coarse aggregate, and the other 13.2 mm aggregate. This results in a longitudinal construction joint and at least two separate placements, which must be reviewed against impacting the construction schedule.

Material, construction, and quality assurance requirements for normal concrete overlays are well established within *OPSS.PROV 930* and *1350*. Concrete is supplied from ready-mix plants certified by the Ready Mixed Concrete Association of Ontario (RMCAO). The use of mobile mixers is prohibited. For remote sites, normal concrete overlays may not be feasible due to travel time from plant to site. Normal concrete overlay requires a waterproofing membrane system as normal concrete is relatively permeable to water and chloride ingress. The overlay increases the dead load on the existing structure.

P2-3.3.2.2 Latex Modified and Rapid Set Latex Modified Overlays

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Latex modified concrete (LMC) consists of Portland cement and aggregates mixed with an organic polymer latex, most commonly styrene-butadiene rubber latex in an emulsion [10]. The addition of latex reduces permeability and provides a better bond to the existing concrete substrate. LMC is typically moist-cured for 48 to 60 hours and then air-cured until the required compressive strength is achieved [11].

Rapid set latex modified concrete (RSLMC) uses high early strength cement for faster strength gain and can reach 20 MPa within 4 hours. It is also referred to as high early strength latex-modified concrete (LMC-HE) and very early strength latex modified concrete (LMC-VES) in literature.

MTO has constructed exposed overlays using LMC, as well as LMC overlays that are waterproofed and paved. Exposed LMC overlay is an option for bridges that cannot support the additional dead load from a normal concrete overlay.

The work is performed by specialized subcontractor, currently from out of province. This overlay method can be used for isolated rehabilitation contracts because it is mixed on site in a specialized mobile mixer. However, there is a cost premium associated with it and requires more lead time.

On projects where LMC overlays are specified, it is recommended to include a trial placement, representative of the actual overlay width to provide the contractor with an opportunity to familiarize themselves with the material. Workers have noted that wet latex-modified concrete feels sticky and is more difficult to finish. Slump must be carefully controlled to achieve the proper slope, especially on structures with slopes or cross-fall greater than 4%. There are stricter limits, compared to normal concrete, on the temperature range over which it can be placed and cured. Proper curing is key to mitigating plastic shrinkage. LMC is more prone to plastic shrinkage cracks due to minimal bleeding, higher paste content, and increased sensitivity to evaporation. The evaporation rate may exceed the bleeding rate of LMC especially if placed in dry and/or windy conditions.

P2-3.3.2.3 Ultra-High Performance Fibre Reinforced Concrete (UHPFRC) Overlay

Ultra-high performance fibre reinforced concrete (UHPFRC) is a cementitious composite material having a minimum specified compressive strength of 120 MPa and enhanced durability characteristics, as defined in *CSA A23.01* Annex U. Other sources have slightly different minimum compressive strength thresholds. Generally, UHPFRC consists of Portland cement, fine sand, silica fume, high-range water-reducing admixture (HRWR), discontinuous steel fibres, and water [12]. Its enhanced durability characteristics stem from its discontinuous pore structure which reduces water, and chloride ingress [12].

The Ministry has successfully used UHPFRC in field-cast joints to connect precast deck panels. This is currently the most popular UHPFRC application in US and Canada [13]. An MTO Designated Sources for Materials List (DSM 9.25.50) is available, containing pre-qualified products and vendors for using UHPFRC in field-cast joints. However, this DSM List is not applicable for overlays.

Aside from using UHPFRC for field-cast connections, most jurisdictions have focused on using UHPFRC in new bridges by optimizing member sizes and thicknesses to save material and to offset its high initial cost compared with conventional concrete [12]. As of 2020, there has been

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approximately 150 UHPFRC overlays installed on bridges worldwide and 17 in the US [14]. The Chillon Viaduct rehabilitation in Switzerland is perhaps the best-known bridge rehabilitation involving UHPFRC overlay; an exposed 45 mm reinforced UHPFRC overlay was installed on the 2.2 km prestressed concrete box girder bridge over Lake Geneva near Montreux to rehabilitate and strengthen the deck [15].

An exposed UHPFRC overlay was constructed in 2024 on the Highway 28 Snake Creek Bridge (site no. 29X-0108/B0) as part of its rehabilitation. The Ministry will monitor and evaluate the performance of this installation and provide future guidance.

P2-3.3.2.4 Other Overlays

Requirements for **silica fume concrete overlays** are specified within *OPSS.PROV 930* and *1350*. Silica fume is a by-product from the production of silicon or alloys containing silicon and is highly pozzolanic. It is added to concrete in a blend of Portland cement and silica fume and makes concrete less permeable compared to normal concrete. Silica fume concrete is prone to shrinkage cracking and requires immediate fog misting during placement and seven days of continuous wet curing. The work is specialized, and silica fume concrete overlays have not been widely used historically on Ministry projects and have not been specified on an MTO contract since 2005.

Polymer concrete consists of aggregates bound with polymer resin. **Polyester polymer concrete (PPC)** is a specific type of polymer concrete using silica and basalt aggregates, an unsaturated isophthalic polyester–styrene resin, silane, and a catalyst that hardens through polymerization. PPC is supplied in bags and drums, mixed on site, and typically imported from U.S. suppliers. As of 2026, MTO has completed one exposed PPC overlay trial and four bridge rehabilitations with PPC overlay topped with waterproofing and asphalt. Performance of the existing PPC overlays will continue to be monitored but given the construction challenges and cracking issues observed to date, The Ministry's Engineering and Materials Office, Concrete Section is currently not recommending PPC overlays.

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P2-3.3.3 Design Requirements

Table 22 provides an overview of the properties and characteristics of the normal concrete, latex-modified and ultra-high performance concrete overlays. Requirements, such as the minimum 28-day compressive strength, permeability, working and curing times etc., are based on Ministry's specifications. It is worth noting that other jurisdictions and literatures may present different values.

Table 22 – Summary of Requirements and Characteristics of Different Overlays.

	Normal Concrete	LMC	UHPFRC
Number of MTO Contracts*	100+	5 - 10	1
Min. 28-day compressive strength	30 MPa	35 MPa	130 MPa

	Normal Concrete	LMC		UHPFRC
Rapid Cl^- permeability at 28-32 days (C)	< 2,500	2,000 – 2,500		< 300
Min. thickness	45 mm	35 mm		25 mm
Working time	~ 1.5 hrs	LMC	~ 1.5 hrs	10 to 60 mins
		RSLMC	< 0.5 hrs	
Min. wet curing	4 days	LMC	24 to 36 hrs	Not Required
		RSLMC	4 to 24 hrs	
To reach 20 MPa	3 to 5 days	LMC	3 to 5 days	2 - 24 hours
		RSLMC	~ 4 hrs	
Require waterproofing	Yes	No**		No
Speciality sub-contractor	No	Yes		Yes
Trial Placement	Not Required	Required		Required
QA Tests	Standard	Extra***		Extra***

**Based on tendered MTO contracts from 2002 to 2022.*
***MTO has constructed exposed and waterproofed LMC overlays.*
****Requires additional tests that are generally not covered under a typical QA Testing Agreement. It is prudent to discuss these requirements with Construction Management staff during detailed design.*

P2-3.3.3.1 Galvanic Corrosion Protection in Overlays

When linear anode systems are embedded in concrete overlays, or other similar horizontal applications, they shall be the 'mortar encased' type to prevent damage to the anodes during placement. A bedding mortar is required.

P2-3.3.3.2 Material Compatibility

Applying an overlay, or any repair material, to an existing concrete structure creates a complex composite system. The existing concrete and new overlay must be compatible to ensure the interface bond strength is not exceeded to achieve long-term performance. Table 23, modified from [16], summarizes properties of repair materials (R) compared to existing concrete substrate (C) to promote effective and durable repairs.

Table 23 – General Requirements of Repair Materials for Compatibility, modified from [16].

Property	Relationship of Repair Material (R) to Concrete Substrate (C)
Strength in compression, tension, and flexure	R ≥ C
Modulus in compression, tension, and flexure	R ~ C

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Coefficient of thermal expansion	R ~ C
Adhesion in tension and shear	R ≥ C
Curing and long-term shrinkage	R ≤ C

When an overlay and concrete substrate with different volume-changing properties experience a significant temperature change, the resulting differences in volume change can increase the risk of failure at the bond interface or within the weaker material [17]. While the individual relationships outlined in Table 23 are desirable, they are not always feasible, and there is no consensus on how these properties interact or whether meeting these criteria ensures compatible performance. The Ministry's standard practice is to use 30 MPa concrete for normal concrete overlays irrespective of the compressive strength of the substrate. The intent of Table 23 is not to deviate from this established practice; rather, it is to emphasize the complex interactions between materials with different properties, to highlight the limitations of our standard practices and their associated risks and uncertainties, so that there is realistic understanding, during the design stage, of the intent and expectation of the repairs.

P2-3.3.4 OPS Specifications and References

Relevant OPS specifications, non-standard special provisions (NSSP) and contract design, estimating and documentation (CDED) chapters related to overlays are summarized in Table 24.

Table 24 - OPS Specifications and References for Overlays.

Means and Methods	OPS Specifications	NSSP (Custodian Offices)	CDED Chapter
Normal Concrete	OPSS.PROV 930	-	
Latex-Modified & Rapid Set Latex-Modified Concrete	-	Yes (Concrete Section & Structures Office)	-
Ultra-High-Performance Concrete	-	Yes (Concrete Section & Structures Office)	-

A comprehensive repository of MTO technical documents can be found on the MTO Technical Publications website.

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P2-3.4 Refacing and Jacketing

P2-3.4.1 Background

Concrete refacing involves placing a layer of new concrete onto an existing surface. It is suitable for repairing components with large areas of deterioration and chloride contamination, significant scaling across their entire surface, or protecting elements that are deteriorating due to low concrete cover. The rate of deterioration is slowed by the increased concrete cover. Refacing can be designed to strengthen a structurally deficient component. The appearance of a refaced surface is more aesthetically pleasing compared to patch repairs.

Concrete refacing of columns is commonly referred to as concrete ‘jacketing’. However, different materials can be used for column jacketing including concrete, steel, or fibre-reinforced polymers (FRP). In this section, references to column jacketing describes partial or full-height encasement of existing columns using concrete. It should be noted that MTO does not use a separate tender item for jacketing columns; the same refacing item is used whenever concrete encasement is specified, regardless of the element.

Except for deck soffits, refacing can be specified for most components exhibiting extensive deterioration. For bridge deck top surfaces, refer to Section P2-3.3 Overlays.

P2-3.4.2 Construction Details

Picture 7 and Picture 8 show a bridge abutment before and after refacing. Control joints not only serve a functional purpose but can enhance aesthetics simultaneously.

Picture 7 – Abutment wall before concrete refacing.

Picture 8 – Same abutment wall after refacing.

Where the top of the refacing is easily accessible, concrete is typically poured from the top through the opening. Otherwise, the ‘form and pump’ method is used. It is important to specify the appropriate refacing tender item reflecting the placement method in the Contract Documents.

Similarly, concrete for column jacketing can be placed either directly from the top or by using the ‘form and pump’ method. When it is placed from the top, the jacket often stops short near the top to allow access for the concrete chute, as shown in Picture 9. A full-height column jacket is more aesthetically pleasing but generally requires the ‘form and pump’ placement method. Consistency amongst columns should be considered to maintain symmetry and ensures that adjacent columns have similar protection.

Picture 9 – Column jacketing stopped just short of top.

P2-3.4.3 Design Requirements

P2-3.4.3.1 Pre-Existing Damage

Refacing can substantially reduce the rate of corrosion but will not stop corrosion damage if chloride contaminated concrete is not removed. The service life of the refacing is influenced by how much chloride is remaining, concrete cover, rebar spacing, and the exposure environment. In areas with severe chloride exposure, consider providing galvanic corrosion protection as part of the refacing. Wide or active cracks in the existing concrete will eventually reflect through in the new refacing and should be repaired first.

P2-3.4.3.2 Surface Preparation

Delaminated concrete should be removed. If more than half the diameter of the reinforcing bar is exposed, concrete should be uniformly removed by 25 mm around the bar. However, for columns where spirals are in contact with concrete behind them, they should be maintained wherever possible. Remaining existing concrete surfaces must either be roughened or removed to a specified depth.

P2-3.4.3.3 Reinforcement and Thickness

For horizontal surfaces, the minimum thickness of the refacing is 50 mm. On vertical surfaces, the refacing should be reinforced with galvanized wire mesh or reinforcing steel equivalent to 0.2% of the sectional area of the refacing in each direction to control shrinkage and thermal cracks. The minimum thickness is 75 mm with wire mesh or 125 mm with reinforcing steel, providing minimum 50 mm cover in both cases.

To anchor the concrete refacing to existing concrete, new dowels should be spaced at intervals no more than 150 mm x 150 mm for wire mesh and 300 mm x 300 mm for reinforcing steel.

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Stainless steel or GFRP may be considered where a minimum cover of 50 mm cannot be provided, or areas where repair would be subjected to salt splashing and severe chloride exposure.

P2-3.4.3.4 Staged Reconstruction

The designer must evaluate and specify whether removal areas must be repaired in stages based on the structural integrity of the element, particularly for columns. In some cases, temporary supports may be required.

P2-3.4.4 OPS Specifications and References

Relevant OPS specifications, non-standard special provisions (NSSP) and contract design, estimating and documentation (CDED) chapters related to concrete refacing are summarized in Table 21.

Table 25 - OPS Specifications and References for Concrete Refacing.

Means and Methods	OPS Specifications	NSSP (Custodian Offices)	CDED Chapter
Concrete Refacing	OPSS.PROV 930	-	-

A comprehensive repository of MTO technical documents can be found on the MTO Technical Publications website.

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P2-3.5 Component Reconstruction

P2-3.5.1 Background

Reconstruction involves partially or completely removing and replacing a component. It may be warranted due to deterioration, insufficient structural capacity, or required modifications to the existing structure, such as a semi-integral abutment conversion.

P2-3.5.2 Construction Details

Removing existing concrete typically involves mechanically breaking it using impact equipment such as chipping hammers, jackhammers, or rig-mounted breakers as detailed in *OPSS.PROV 928* (refer to Section P2-3.1 Concrete Removal), or alternatively removing it through controlled saw-cutting.

Removal details must account for access, equipment restrictions outlined in *OPSS.PROV 928*, and the resulting impacts on productivity. Site-specific constraints, such as narrow or congested work areas, can further reduce production rates.

In the vicinity of concrete that is to remain, removal is generally limited to using chipping hammers or saw-cutting. Chipping allows existing reinforcing bars to be exposed, allowing new bars to be lapped. Saw-cutting, by contrast, severs the existing bars, requiring any new bars to be dowelled instead. Although saw-cutting and dowelling is typically faster, the use of dowels is subject to specific limitations. These dowelling restrictions and design considerations are described in Section P2-3.15 Post-Installed Adhesive Anchors into Concrete (Dowels).

P2-3.5.3 Design Requirements

P2-3.5.3.1 Replacing Superstructure Components

CSA S6-25 CHBDC codified requirements for non-replaceable components, which must be designed for the full service life of the structure, a minimum of 75 years. Superstructure components fall within this category. Although this requirement is new in the 2025 edition of the code, MTO bridges constructed since the mid-1980s using the Ministry's modern waterproofing system have demonstrated strong long-term performance. Based on observed trends, these bridges are expected to satisfy the new code requirement, with their superstructure components achieving their full design service life.

In cases where replacement of superstructure components is being considered, replacing the entire superstructure is preferable to replacing only the deck, particularly for thin slab-on-girder structures. This approach avoids leaving behind components that may be code-obsolete under the current standards, ensuring that all components of the reconstructed superstructure comply with the same, current design requirements. Replacing only the deck can result in mismatched service lives between new and existing components and increase long-term uncertainty regarding the condition and performance of the remaining girders or components. Deck removals are very labour-intensive, with a high risk of damaging remaining elements. Full superstructure replacement simplifies construction, reduces uncertainty, modernizes all components to the same code standard, and provides a unified service-life reset for the superstructure.

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P2-3.5.3.2 Replacing Substructure Components

Substructure reconstruction is generally limited to wingwalls, ballast walls, and partial reconstruction of abutment walls, often carried out as part of a semi-integral abutment conversion. In such cases, it is advantageous to capitalize on the improved access to reconstruct components that are otherwise inaccessible. This reduces construction uncertainty and helps ensure alignment of service life across all elements.

P2-3.5.3.3 Coupling Stainless Steel and Carbon Steel

Although coupling dissimilar metals in the presence of an electrolyte (e.g., de-icing chemicals) can raise concerns about galvanic corrosion, research have shown that pairing stainless steel and carbon steel in concrete does not increase corrosion risk. As described in *CSA S6.1-25 CHBDC Commentary* Clause C8.11.2.5, stainless steel has been shown to induce negligible galvanic corrosion to adjacent carbon steel, and studies indicate that a stainless-steel–carbon-steel rebar pair does not exhibit any greater corrosion risk than a carbon-steel–carbon-steel pair when embedded in concrete.

P2-3.5.3.4 Interface between Existing and New Concrete

The interface between new and existing concrete can present an elevated corrosion risk due to differences in chloride content, permeability, and alkalinity. These contrasts can create electrochemical imbalances that form a corrosion cell, where reinforcing steel in the newer, lower-chloride, higher-alkalinity concrete acts as the cathode, and reinforcing steel in the existing chloride-contaminated concrete becomes the anode, accelerating corrosion of the existing reinforcing bars.

Given these vulnerabilities, the interface, between the new and existing concrete, is a critical area in rehabilitation projects, and the selected corrosion protection strategy shall be suitable for the exposure zones defined in *CSA S6-25 CHBDC*. For reconstructed interfaces exposed to deicing salts, specifically splash zones (D1–D3), the use of galvanic corrosion protection measures shall be considered. However, an interface between an existing deck and a new deck that is fully protected beneath a waterproofing membrane does not require additional galvanic protection measures.

In addition, any surface irregularities or poor bonding at the interface can trap moisture, further promoting corrosion activity. Adequate roughening helps ensure proper bond and reduces the potential for leaking through construction joints, which are inherently vulnerable. For concrete interfaces specifically designed to transfer shear through bond, the designer shall note the specific roughening requirements (area and amplitude) on the drawings.

The Ministry does not use waterstops between concrete pours due primarily to difficulties associated with accurate placement and poor historical performance.

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P2-3.6 Concrete Crack Repairs

P2-3.6.1 Background

Nearly all concrete components are subject to fine and hairline cracking, and in most cases, this cracking is not a cause for concern and no treatment is needed. Before cracks are designated for repair, the designer needs to determine if remedial measures are necessary and whether an effective repair is feasible and economical. Cracks should be repaired when the structural load carrying capacity or durability is affected. The cause of the cracking should be determined and eliminated, if practical.

P2-3.6.1.1 Causes of Cracking

The cause(s) of cracking must first be determined to select the most suitable repair strategy. The investigation should also ascertain whether the mechanism that caused the cracking is still active or will reoccur in the future. Different types of concrete cracking and a description of the associated deterioration mechanisms can be found in *CSA S6.1-25 CHBDC Commentary* Clause C2.4.2.

P2-3.6.1.2 State of Activity and Extent of Cracking

Cracks are categorized as active or dormant, depending on whether the mechanism that caused the cracking is still active or not. Active cracks are increasing in number and size because the underlying mechanism is still active. Active cracks may be repaired with a flexible material to allow for movement, or the cause of the cracking must be eliminated. Dormant cracks have stabilized and remain constant in number and size and can be filled using a more rigid material.

The extent of cracking influences the overall repair strategy. It may not be economical or necessary to repair isolated cracks. On the other hand, widespread cracking may be indictive of fundamental structural issues (meant in the broadest sense here), or due to material defects, such as alkali–aggregate reaction (AAR) or delayed ettringite formation (DEF), that cannot be resolved with crack repairs alone.

Monitoring and mapping may be required to determine the extent of cracking and their state of activity. Movement can be monitored with a crack measuring device to record crack displacement and rotation. Width and depth can be measured using a feeler gauge, crack comparator, or by taking concrete cores. Testing requirements are described in Section P1-9.7 Concrete Cores and Physical Testing.

P2-3.6.2 Construction Details

P2-3.6.2.1 Crack Injection

This method of crack repair involves cleaning the cracks, sealing the surface, installing entry, and venting ports, and pumping epoxy or polyurethane resin into the cracks using a positive displacement pump. Construction requirements for crack injection repairs are detailed in *OPSS.PROV 932*. It should be noted that cathodic protection may not function properly in repaired areas as the resins may insulate the underlying reinforcement.

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Epoxy

Epoxy resin is the most common adhesive used for concrete crack repairs, [18]. It is suitable for cracks up to 5 mm wide and is the most suitable method for restoring structural strength and water tightness of a component, provided that the cause of cracking is eliminated and where there is no high waterhead (e.g., it is not the right material for repairing a culvert below the water table). Epoxy is not effective for active cracks as new cracks will likely occur adjacent to the injected crack.

Polyurethane

The polyurethane resin should be selected for active cracks where it is not necessary to restore structural strength. As the term polyurethane is sometimes applied to prepolymers, which can be rigid, it is important that the product selected is flexible. A 100% solid, water compatible hydrophobic polyurethane elastomer is recommended for most applications. The product should be able to displace water in the cracks and have a good bond to wet or dry concrete [18].

For applications where there is significant leakage, a hydrophobic flexible foam prepolymer may be more suitable. However, the Manufacturer should be consulted to provide advice on materials and techniques for sealing cracks that are leaking due to high waterhead.

Rout and Seal

This method of crack repair involves routing a crack to create a chase and sealing with a sealant. This method is typically used to repair cracks in concrete pavements. A flexible sealant is generally specified to accommodate thermal movements.

Hot-poured rubberized joint sealing compounds are used on horizontal surfaces, approved suppliers and products are listed under the Ministry's *DSM List 3.20.45*. Cold-applied joint sealing compounds are used on vertical surfaces or on horizontal surfaces that are not to be waterproofed. The practice of routing and sealing cracks on vertical surfaces is not common.

Gravity Fill

This method involves pouring sealant directly over cracks (without routing) or applying to an entire area using squeegees, rollers, or brooms. Gravity filling, sometimes referred to as crack healing, for cracks 0.1-6 mm in width is typically done with low (<2000 cP) or very low viscosity (<1000 cP) epoxy and works best to seal the surface of cracks as a durability measure. When deeper crack penetration is required to restore structural capacity, pressure injection should be specified.

When the gravity feed method is used on waterproofed bridge decks, excess sealant shall be ground off to meet the surface preparation requirements for waterproofing and to permit adequate bond between the waterproofing membrane and concrete deck. For exposed decks, surface friction characteristics of the sealant, including any grit course added, need to be evaluated when it is applied over an entire area. Gravity filling is not covered under *OPSS.PROV 932*, and it is not typically specified under MTO contracts.

P2-3.6.3 Design Requirements

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REHABILITATION SELECTION

Concrete components frequently exhibit some degree of cracking. In most cases, cracks in mildly reinforced concrete are not cause for concern and repairs are not necessary. However, cracking in prestressed concrete elements must be carefully evaluated to determine the root cause.

Crack sealing for durability should consider the exposure zone, as defined in *CSA S6 CHBDC*, and the impacts of cracking over the service life of the component. Determining whether the cracks are active or dormant is essential for selecting the appropriate method and material for repair. Accurately diagnosing the causes of cracking is crucial and requires a comprehensive understanding of the element's construction and the forces acting on it, which may involve monitoring or a structural analysis.

Active cracks are often a symptom of an underlying issue, and it is more effective to identify and address the root cause to eliminate the problem altogether. Active cracks will likely continue to form and propagate unless the driving forces or actions causing the cracking is addressed. Crack repairs should not be specified indiscriminately across an entire structure, as this is both costly and unnecessary. Before deciding to repair cracks, the designer must assess the causes of cracking and their associated risks, such as exposure to de-icing chemicals, concrete cover, type of reinforcing steel etc., and weigh these factors against the potential benefits of repair. The causes of cracking and the justification for repair must be documented during design. The decision to repair cracks in a specific structural element should not be left to the Contract Administrator.

Typical requirements and applications for crack injection and rout and seal are listed in Table 26.

Table 26 – Summary of Typical Requirements for Different Concrete Crack Repair Methods and Materials.

	Crack Injection using Epoxy Resin	Crack Injection using Polyurethane Resin	Rout and Seal using Cold or Hot-Applied Joint Sealants
OPSS.PROV 932 Material Requirements	Type I and IV, Grade 1, Class B and C per ASTM C881	100% Solid	DSM 3.20.45 for Hot-Applied Sealants Type S or M, Grade NS, Class 25, Use T and M per ASTM C920, for Cold-Applied Sealants.
Surface	■ Horizontal ■ Vertical ■ Overhead	■ Horizontal ■ Vertical ■ Overhead	■ Horizontal ☐ Vertical ☐ Overhead
Crack Width (1)	0.3 – 5 mm	0.3 – 5 mm	> 0.3 (5)
Restores Structural Strength	Yes (3)	No	No
Cracks Exposed to Moisture (2)	Yes	Yes	Yes
Cracks with Seepage	No (4)	Yes	No

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Active Cracks	No	Yes	Yes
Dormant Cracks	Yes	Yes	Yes
Surface to be Waterproofed	No	No	Yes (6)

[1] Cracks less than 0.3 mm do not require treatment.
[2] Cracks not exposed to moisture do not require treatment as corrosion of the embedded reinforcing steel will be minimal.
[3] If the cause of cracking is not eliminated, new cracks will likely develop adjacent to injected cracks.
[4] Epoxy can be used to inject cracks with minor seepage. However, a flexible hydrophobic polyurethane resin should be used for cracks with major seepage under high waterhead.
[5] Cracks wider than 5 mm may require to be repaired as a patch, prior to routing and sealing.
[6] If the surface is to be waterproofed with a hot applied rubberized waterproofing membrane, cracks less than 1.0 mm wide do not need to be treated prior to waterproofing. Cracks more than 1.0 mm wide should be repaired first using a hot applied sealant prior to waterproofing.

P2-3.6.4 OPS Specifications and References

Relevant OPS construction specifications, non-standard special provisions (NSSP) and contract design, estimating and documentation (CDED) chapters related to concrete crack repair are summarized in Table 27.

Table 27 - OPS Specifications and References for Concrete Crack Repairs.

Means and Methods	OPS Specifications	NSSP (Custodian Offices)	CDED Chapter
Crack Repair	OPSS.PROV 932	-	B-932

A comprehensive repository of MTO technical documents can be found on the MTO Technical Publications website.

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P2-3.7 Concrete Coating

P2-3.7.1 Background

Concrete coatings and concrete sealers are used for a wide variety of applications in concrete construction. Concrete sealers can be used as protective coatings to enhance the resistance of concrete to moisture, water, and chloride ion penetration. Other coatings can be applied for architectural purposes. Sealers can be applied onto the exposed surfaces of new and existing concrete.

P2-3.7.1.1 Types of Sealers

The Ministry's Engineering Materials Office, Concrete Section maintains and periodically updates a list of approved concrete sealers. The list is available upon request and includes different types of penetrant-pore blocker and penetrant-pore liner sealers.
Penetrant-pore blockers react chemically with calcium hydroxide in concrete to fill the pores, thereby reducing the concrete's permeability. Silicates (densifiers) like sodium, lithium, or potassium silicates are the most popular pore blockers.

In contrast, penetrant-pore liners form a hydrophobic lining within the pores without blocking them, allowing vapours and gases to escape. Silicon-based products, such as silane and siloxane, are the most popular pore liners and are the ones most used on MTO projects.

Pigmented sealers typically consist of a silicon-based primer coat with a pigmented acrylic topcoat. The pigmented topcoat is used for aesthetic reasons to achieve consistent colouration and appearance. While they remain breathable, their permeability is comparatively reduced.

Anti-graffiti coatings are applied to areas that experience high incidence of unwanted graffiti in highly visible locations that facilitate ease of removal through pressure washing. MTO evaluated two basic categories of anti-graffiti coatings, those with a urethane topcoat and those with a siloxane topcoat.

Some sealers contain corrosion-inhibiting additives and are referred to as surface-applied corrosion inhibitor (SACI) sealers. The inhibitors penetrate the concrete and form a protective layer around the embedded reinforcing steel.

P2-3.7.2 Construction Details

Sealers are typically applied to the concrete surface using low-pressure sprayers or hand rollers as per the manufacturer's recommendations. Proper surface preparation is crucial for the durability of the sealer, ensuring the concrete surface is clean, adequately roughened, sufficiently dry, and within the acceptable temperature range. Sealers require the concrete to be adequately curing and typically 28 days is needed prior to application, otherwise they can suffer from premature bond failure. This often requires contract provisions for scheduling the work in a subsequent construction season where environmental conditions are suitable for application.

Manufacturers generally specify an application rate that correlates to the desired sealer film thickness. A thin layer may not provide adequate coverage and protection, while a thick layer can lead to poor adhesion, resulting in peeling or flaking over time. Additionally, air and moisture

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trapped beneath a thick layer of sealer can cause bubbling and blistering. Whether applied too thick or too thin, it can lead to an uneven and inconsistent finish, particularly effecting the appearance of pigmented sealers.

Sealers require periodic reapplication to remain effective. Traffic control and access requirements for reapplication should be considered. Reapplication involves full removal of the existing coat. This is particularly important for pigmented sealers otherwise the previous coat can block concrete pores, leading to accelerated flaking and blushing (white spots) in the fresh acrylic coating due to trapped moisture.

Expected service life of penetrating silane sealer is 5 to 10 years depending on exposure and the internal pH level of the concrete, assuming the concrete does not crack during this period.

Pigmented acrylic sealer may exhibit flaking within 5 to 10 years depending on workmanship and exposure conditions. However, its aesthetic appeal from a distance is generally still acceptable.

Penetrating and pigmented sealers are not effective for sealing cracks. New cracks would effectively break the hydrophobic lining in pore lining penetrating sealers and cause pigmented sealers to flake and peel around them.

P2-3.7.3 Design Requirements

P2-3.7.3.1 New Concrete

Concrete produced and placed in accordance with the Ministry’s standard specifications is inherently durable and does not require additional protective measures such as sealers. For this reason, on rehabilitation projects where components, like barrier walls, are fully replaced with new concrete, concrete sealers should be avoided unless approved by the Head of the Regional Structural Section.

Sealers should not be used on traffic surfaces such as bridge decks. In Ontario, most highway bridge decks are protected with a hot-applied rubberized asphalt membrane waterproofing system, which are then paved over with asphalt (refer to Section P2-3.11 Waterproofing).

P2-3.7.3.2 Existing Concrete

Surface-Applied Corrosion Inhibitors

SACI sealers may enhance durability, but they should be applied judiciously and as a supplemental measure, such as delaying the onset of corrosion, rather than as a primary treatment for concrete that is already actively deteriorating.

SACI sealers are generally not suitable for highly carbonated concrete, concrete with cover exceeding ±60 mm, concrete with chloride concentrations greater than 0.1–0.2% by mass of concrete at the reinforcement level, or low-permeability concrete where the inhibitors cannot adequately migrate to the steel.

Due to these limitations, structures already exhibiting visible deteriorations are unlikely to benefit from SACI sealers. However, they may be effective as a supplemental/preventative measure

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where concrete cover is low, permeability is relatively high, but chloride contamination has not yet reached elevated levels.

IRCI Guide 510.2 (Guide for Use of Penetrating Surface Applied Corrosion Inhibitors for Corrosion Mitigation of Reinforced Concrete Structures) provides guidance on the selection, evaluation, application, and performance assessment of penetrating surface-applied corrosion inhibitors for mitigating corrosion in reinforced concrete structure.

SACI sealers function through different inhibition mechanisms, including anodic, cathodic or ambiodic (mix). The SACI sealers commonly used on MTO projects are organo-functional silane products, belonging to the ambiodic category. Their effectiveness cannot be readily assessed in the field and requires laboratory analysis, such as mass spectrometry, to verify inhibitor presence and performance.

Pigmented Coatings

Pigmented coatings should be considered for concrete surfaces where there is a genuine need for consistent appearance. Bridges are significant structures in the built environment, and a high-quality finished appearance is important following significant capital expenditure to promote public confidence. Pigmented sealers should be considered for bridge and retaining wall rehabilitations where patching and repairs will lead to an uneven visual appearance in highly visible areas on Level 1-High or Level 2-Medium aesthetic classification structures. Their use can enhance the visual appearance of the existing structure, particularly when the surface has a mix of new and existing concrete.

It is not necessary to use pigmented sealers to coat an entire structure or to coat all elements of a specific type within a structure. Only surface areas that require aesthetic attention should be coated with a pigmented sealer.

Highly visible locations where pigment sealers should be considered, include:

- Where rehabilitation results in old concrete adjacent to new concrete.
- Where there are numerous patches along the surface.
- Where the barrier wall at the expansion joints has been replaced, while the remainder wall has not.

Picture 10 and Picture 11 show the contrast in application between bridges left untreated and sealed with a pigmented sealer respectively.

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Picture 10 - Unsealed rehabilitated deck end.

Picture 11 – Rehabilitated deck end with pigmented sealer applied.

Pigmented sealers should not be used on surfaces not in plain view or sites that are not sensitive to aesthetic appearance or surface colouration. Pigmented sealers should not be used to conceal construction issues, instead of addressing the deficiencies. Pigmented sealers will not conceal bug holes, voids, cracks, or poor surface finishing. Surface defects should be repaired prior to applying sealer. Sealed surfaces, like uncoated surfaces, can be stained over time by active corrosion beneath the surface, and by runoff from other areas.

Colour should be specified based on *AMS-STD-595 (Colors Used in Government Procurement, SAE International)*. The Ministry has typically specified colours 26493 and 26373 for concrete structures, with the latter generally applied to surfaces predominantly in shade. Where close colour matching is of particular concern, test panels should be included as part of the work to verify acceptability.

Pigmented sealers are two-part systems consisting of a penetrating silane base coat followed by a pigmented acrylic topcoat. Although pigmented sealers are not primarily intended to improve durability, the silane base coat does provide some beneficial protection. The acrylic topcoat will eventually flake, but the onset and extent of flaking vary depending on application quality and exposure. In some instances, noticeable flaking has occurred within five years, while other sites continue to perform adequately 15+ years after installation. While minor flaking is visible up close, it is generally not noticeable at a distance. Touch-ups may be carried out during the service life of the coating; however, full reapplication requires complete removal by abrasive blast cleaning to provide the required adhesion for the new coating prior to re-installation.

When aesthetics is the only purpose, only the pigmented topcoat needs to be reapplied. If an appropriate colour is chosen, a structure with weathered sealer is more uniform and pleasing than an unsealed surface and may not warrant reapplication until the next rehabilitation. However, if the coating system is intended to provide the protection of a sealer, along with improved appearance, then both the penetrating silane and topcoat should be reapplied.

Anti-Graffiti Coating

Anti-graffiti coating should only be considered on elements with existing graffiti that are visible to the public. Consideration of anti-graffiti coatings application requires co-ordination with maintenance staff to ensure the appropriate cleaning methods are communicated.
Anti-graffiti coating works by creating a protective non-stick surface that makes it easier to remove graffiti, such as using a pressure washer, instead of needing to abrasive blast clean. It is not

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meant to paint over existing graffiti. The anti-graffiti coating must be able to achieve cleanability level 1 based on the latest *ASTM D7089 (Standard Practice for Determination of the Effectiveness of Anti-Graffiti Coating for Use on Concrete, Masonry and Natural Stone Surfaces by Pressure Washing)*.

Ottawa and other cities have engaged local artists to paint murals on their structures, such as the abutment walls of the Highway 417 Bank Street Underpass. It has deterred unwanted graffiti, as graffiti artists generally respect other artists' work.

P2-3.7.4 OPS Specifications and References

Relevant OPS construction specifications, non-standard special provisions (NSSP) and contract design, estimating and documentation (CDED) chapters related to concrete sealers are summarized in Table 28.

Table 28 – OPS Specifications and References for Concrete Sealers

Means and Methods	OPS Specifications	NSSP (Custodian Offices)	CDED Chapter
Concrete Sealer	-	Yes (Concrete Section)	-

A comprehensive repository of MTO technical documents can be found on the MTO Technical Publications website.

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P2-3.8 Cathodic Protection

P2-3.8.1 Background

Corrosion is the deterioration of a material or its properties due to unwanted chemical interactions with its environment. Cathodic protection specifically targets electrochemical corrosion, some understanding of basic electrochemistry is essential for its control.

The international standards for cathodic protection of reinforced concrete structures are:

- *ISO 12696 – Cathodic protection of steel in concrete*,
- NACE *AMPP SP0290-2019 Impressed Current Cathodic Protection of Reinforcing Steel in Atmospherically Exposed Concrete Structures*
- *NACE AMPP SP0216-2023 Galvanic Cathodic Protection of Reinforcing Steel in Atmospherically Exposed Concrete Structures*.

The standards describe the two different measures of cathodic protection: impressed current cathodic protection and galvanic anodes. Both systems require monitoring provisions to determine performance and comply with the standards.

Cathodic protection works by disrupting the corrosion cell and forcing the structure to act as the cathode of a direct current circuit. In an electrochemical cell, oxidation, and thus corrosion, occurs at the anodic sites. By applying a more negative potential to the steel, the potential of the cathodic sites shifts (polarize) toward those of the anodic sites, reducing corrosion current. When the reinforcing steel is polarized to a potential that is at least as active as the most active anodic area on the structure, the corrosion process shifts away from the reinforcing steel and onto the designated protection anode.

Cathodic protection is not applicable to reinforced concrete containing electrically conductive fibres (e.g., carbon or steel).

The terms cathodic prevention and cathodic protection are sometimes used interchangeably, but they differ in timing and purpose.

Cathodic prevention is applied to new structures, or to structures in service where chlorides have not yet reached levels sufficient to depassivate the reinforcing steel. Its purpose is to delay corrosion by maintaining protective potentials, so future chloride exposure does not trigger corrosion.

Cathodic protection is applied to structures where corrosion has already begun, with the goal of reducing the ongoing corrosion rate by polarizing the steel to meet specified protection criteria.

It is important to carry out necessary sampling and testing so that the selected cathodic protection approach is appropriately designed for its intended role, whether preventing corrosion initiation or controlling corrosion that is already occurring.

P2-3.8.2 Construction Details

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There are different types of corrosion protection systems, including electrochemical treatments, impressed current cathodic protection (ICCP) and sacrificial corrosion protection (SACP) systems.

Electrochemical treatments change the chemistry of the concrete around the reinforcement to facilitate passivation of the reinforcing steel. Electrochemical Chloride Extraction (ECE) was first used on an experimental basis in 1989 on several bridges and documented in the report MERO-001 [19]. ECE is not currently used by the MTO and is typically only applied in specialty applications, such as when concrete removal and repair methods are not permitted for historical preservation of the original concrete.

Impressed current cathodic protection uses an external power supply and inert anodes to distribute DC current to the steel. This may be directly wired, connected to an embedded substrate, or through a conductive coating. By driving the steel to a more negative potential, the system forces the steel to act as the cathode of the electrochemical cell. The Ministry currently maintains approximately 15 bridges with impressed current cathodic protection, although no new systems have been implemented in the past decade or more.

Sacrificial corrosion protection uses more active metal anodes to deliver galvanic current to the reinforcing steel, causing the sacrificial metal to corrode in place of the steel. This is the most common form of cathodic protection used in Ministry rehabilitation projects. The Ministry has used a variety of sacrificial anode systems, described in greater details below.

P2-3.8.2.1 Sacrificial Corrosion Protection

Sacrificial anode systems have inherently limited current output and therefore may not always be the most suitable protection method. The example in Picture 12 is an embedded anode system and Picture 13 is a substructure treated with thermally sprayed zinc.

Picture 12 – Embedded anodes.

Picture 13 – Thermally sprayed zinc on substructure.

Embedded Discrete Anodes

There are different anode products which may be embedded in the concrete surface, but they have typically been used for different purposes.

- **Linear Anodes**
  The Ministry has used sacrificial linear anodes along reconstructed deck cantilever, overlays and refacing as a means of corrosion control. The monitoring of some sites with linear anodes has shown that the depolarization criterion from the standards can be achieved.

- **Discrete Anodes in Patches**
  The Ministry carried out long term performance monitoring of discrete zinc anodes in barrier wall patches between 1999 and 2009, finding that the 100mV depolarization criteria was not met when tested at 2 and 5 years respectively. While anode suppliers have updated the design of their products, the Ministry does not yet have long-term performance data for the updated products, and widespread monitoring is generally impractical for small patches. In addition, exposure conditions can vary significantly resulting in different levels of contamination and moisture content in concrete. Discrete anodes are also limited by their small zinc mass (typically contain only 40–350 g), making it challenging to achieve and maintain the required current density. By comparison, linear anodes can contain up to 2500 g/m of zinc.

Thermal Sprayed Anodes

Thermal sprayed anodes (TSA) are surface applied and are generally used to target specific areas with higher corrosion risk. Zinc is applied either by flame spray or electric arc-spray techniques. Flame spray uses an acetylene-oxygen flame to melt continuous-fed zinc wire that is then sprayed onto the substrate. Electric arc-spray uses two feedstock wires that act as electrodes. The wires are advanced to meet at an atomizing stream where direct current is applied and an arc forms, causing the tips to melt. A stream of compressed air flows across the arc zone and propels the molten droplets onto the substrate, [20]. A thermally sprayed anode applied to a substructure is shown in Picture 13.

Surface Mounted Anodes

Surface mounted anodes are secured to the exterior of a component to provide corrosion control or cathodic protection and exists in different forms depending on the material or component to be protected.

P2-3.8.3 Design Requirements

Cathodic protection of reinforced concrete elements should be considered when conventional repair techniques, such as concrete removal and repair are not possible or practical. Situations where the structural integrity of the existing component may be compromised with removals, such as prestressed components or highly loaded compression elements are typical examples.

When evaluating the need for a cathodic protection system, the scope and requirements of the structural inspection should be established with guidance from a qualified independent NACE/AMPP Cathodic Protection Specialist (CP4) to ensure that all information necessary for proper system design is collected. The inspection must consider defects, features of the structure, and its surrounding environment that may influence the performance of cathodic protection. Necessary concrete sampling and testing shall be carried out such as chloride

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concentration and depth, carbonation depth, and concrete cover; reinforcing steel size and layout and confirmation of electrical continuity shall also be determined.

The final design must be completed by an engineer who holds the NACE/AMPP CP4 cathodic protection specialist certification independent from the supplier. The design assumptions, the intended service life of the system, and performance criteria over its service life, must be clearly documented in design. Monitoring requirements shall be determined during detailed design. Omission of monitoring requires approval by the Head of the Structural Section.

All details of embedded linear anodes shall be specified in the contract documents, including anode size and zinc content, spacing, and connection details.

Thermally sprayed anodes shall clearly specify limits of application, zinc thickness and connection details. The applied thickness is typically 300um, however the AWS standard *C2.20/C2.20M Specification for Thermal Spraying Zinc Anodes on Steel Reinforced Concrete* considers 500 um an upper limit in special cases for longer service life. It is important to note however that the circuit resistance increases as corrosion product develops decreasing the efficiency of the anode. More effective protection can be achieved by replacing the TSA at regular intervals. Installations larger than 100m2 shall include isolated zones completed with embedded reference electrodes for monitoring purposes. All monitoring requirements shall be detailed in the contract documents.

The Ministry does not endorse the use of embedded discrete galvanic anodes in concrete patch repairs or surface mounted anodes at this time.

P2-3.8.3.1 Performance Criteria

The effectiveness of cathodic protection is measured by the difference in electric potential between the polarized steel and its natural corrosion potential. This is referred to as the "depolarization threshold" and ISO 12696 requires a potential decay of at least 100 mV over a maximum 24-hour period. The NACE/AMPP standard requirements are similar. While others have argued for a lower threshold [21] this has not been adopted by either the ISO or NACE standards.

The degree of polarization is influenced by the current density. Cathodic protection systems typically employ much higher current densities, an order of magnitude, compared to cathodic prevention systems. ISO 12696 cites 0.2 to 2 mA/m^2 current density range for cathodic prevention and 2 to 20 mA/m^2 for cathodic protection on existing corroded structures with uncoated steel.

The effectiveness of cathodic protection measures is influenced by existing conditions and environmental exposures, such as chloride content, pH level, temperature, oxygen content, humidity, reinforcing steel density etc., and the complex interactions of these factors.

It is possible to apply too high of a cathodic protection current to materials; a common example is prestressing wire. High cathodic protection current density may cause hydrogen embrittlement, increase corrosion, lead to excessive acid/chlorine generation, or cause acidification damage to cement-based anodes.

P2-3.8.4 OPS Specifications and References

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REHABILITATION SELECTION

Relevant OPS specifications, non-standard special provisions (NSSP) and contract design, estimating and documentation (CDED) chapters related to concrete removal are summarized in Table 29.

Table 29 – OPS Specifications and References Related to Cathodic Protection.

Means and Methods	OPS Specifications	NSSP (Custodian Offices)	CDED Chapter
Anodes, Embedded Galvanic Type	-	Yes (Structures Office)	-
Thermally Sprayed Anode	-	Yes (Structures Office)	-

A comprehensive repository of MTO technical documents can be found on the MTO Technical Publications website.

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P2-3.9 Expansion Joints

P2-3.9.1 Background

Expansion joints in bridge decks accommodate movements resulting from temperature changes, concrete creep, and shrinkage. They also serve to prevent water, often contaminated with de-icing and corrosive chemicals, from leaking onto the substructure causing damage. Over time, expansion joints frequently deteriorate due to broken seals, seized movement resulting from accumulation of debris and degraded rideability caused by broken armouring, steel plates, or bolts.

Expansion joints are often an ongoing maintenance problem and consideration should be given to eliminating expansion joints at abutments with semi-integral abutments, and over piers with either flexible link slab or semi-continuous details if modifications are feasible and can be constructed economically. Development of flexible link slab to eliminate existing expansion joints over piers for steel girder bridges are documented in Bridge Office Report BO-01-01 [22], including its first use in 1987 on the Gardiner Expressway, and subsequent projects on Ministry bridges.

In some cases, temporary bridging systems (i.e., 'bridging plates') are used to facilitate replacing modular and conventional expansion joints where long-term closures are not feasible. This is described more in Section P2-3.17 Deck Top Rehabilitation using Short-Term Closures.

P2-3.9.2 Construction Details

P2-3.9.2.1 Types of Expansion Joints

The Ministry uses several types of expansion joints, including strip seal, sliding plate, finger plate and modular expansion joints. Detailed guidance and descriptions of each joint type are provided in Section 13 of the *Structural Manual*. Some considerations are highlighted below, with emphasis on rehabilitation.

Strip Seal Joints

Strip seal expansion joints utilize a strip of neoprene to span the gap and are, by far, the most common type of expansion joints in Ontario. The types of strip seal joints are differentiated by the way the seal is secured, refer to Figure 13 and Figure 14.

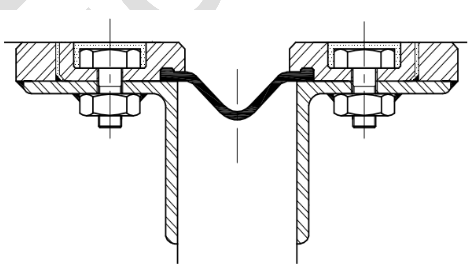

Figure 13 – Example Type A strip seal joint.

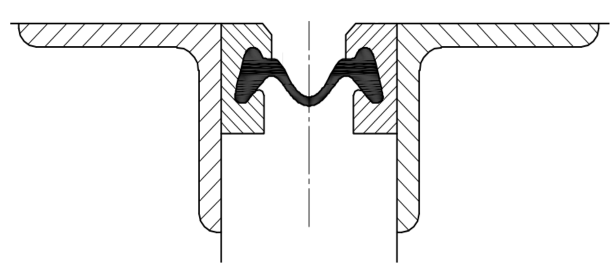

Figure 14 – Example Type C strip seal joint.

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The seal in Type A joints is secured in place with clamping bars, as illustrated in Figure 13. These joints can accommodate movements of up to 75 mm. They are detailed to facilitate direct access to the clamping bolts and seal, replacing the seals is intended to be a quick and easy process, minimizing future lane closures. However, in practice, replacing seals in older Type A joints has often been difficult and time consuming because the bolts were frequently stripped, and the clamping bar was sometimes broken. To improve their performance and facilitate easier seal replacements, several changes were introduced as part of the August 2015 update to Structural Standard Drawing (SSD) 0113.0012 'Strip Seal Expansion Joint – Type 'A'', as shown in Figure 15.

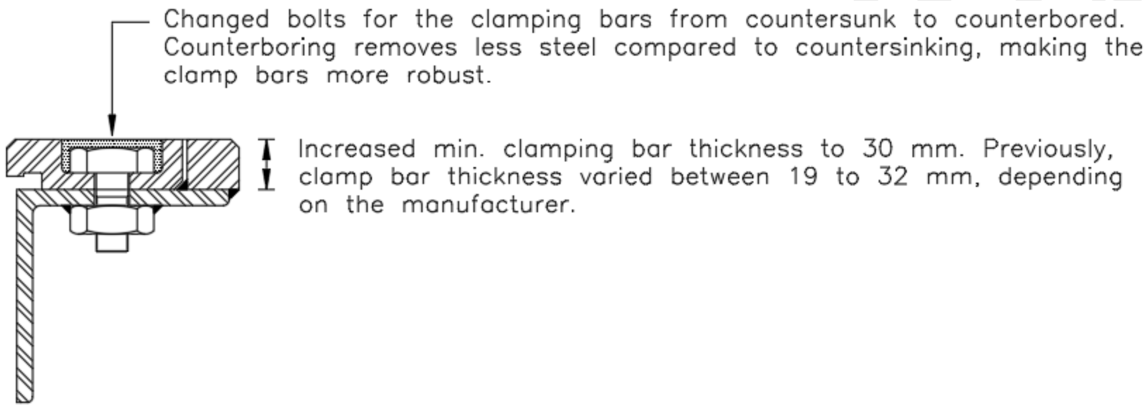

Figure 15 – Changes made to SSD 0113.0012 in the August 2015.

The strip seal is press fitted and secured in retainers in Type C strip seal joints, as illustrated in Figure 14. Type C joints can accommodate movements up to 65 mm. They are cheaper, but their seals and retainers are more difficult and time consuming to replace. The general details of Type C joints have not changed much over the years.

For both Type A and C joints, seals are available in different shapes. Some new seals may not be compatible with existing retainers, and older shapes may no longer be manufactured. It is important to check the seal's availability and compatibility, especially for seal replacements and partial-length reconstruction & extension of existing joints.

Where the seal is damaged but steel armouring and concrete in end dams are in good condition, seal replacement should be specified. Sometimes, a nut needs to be welded to an existing bolt to remove it. Often the threads of existing bolt holes are stripped so the holes need to be enlarged and tapped with new threads for the new bolts. While older details can make it more challenging and time consuming to replace seals, it is generally always feasible.

Existing expansion joints without steel armouring should be replaced with an armoured joint.

Minimum joint anchorage requirements are outlined in the Joints and Bearings section of the *CSA S6 CHBDC*. Mechanical friction-type anchor bolts or anchor bolts embedded in a sleeve of grout have been found to be unsatisfactory and should not be used. All existing black or epoxy coated reinforcing steel in the expansion joint blockout should be replaced with stainless steel in accordance with the Ministry's corrosion protection policy outlined in the latest *Structural Manual*.

Polystyrene foam and other formwork materials must not be left in place after construction. If polystyrene foam or other formwork has been left at an abutment or pier, it must be removed, as it traps moisture and can accelerate deterioration of the concrete.

Sliding ('Finger') Plate Joints

Sliding plate joints can accommodate movements between 75 to 120 mm. These joints consist of a sliding plate extending across the joint opening. Crenellations at the end of the sliding plate overlap with those of a fixed plate ('stop plate') on the other side of the joint to provide a smooth ride for vehicles. These joints have a Type C seal and retainers below the sliding plates.

Compared to strip seal joints, sliding plate joints are more difficult to construct. The alignment of the sliding plate joint is critical; the horizontal sliding surface must be planar with the sliding plate. The sliding plate must bear and slide on the armouring plate otherwise, it can lead to maintenance issues and premature failure of the bolts.

Modular Joints

A modular joint consists of seals, separation beams, and support bars. Modular joints can accommodate more than a metre in movement by varying the number of seals. Wheels bear directly on the separation beams between the seals. The separation beams are supported by steel support bars, spaced at intervals, each having its own support box at the ends. The support bar is fixed in a short support box in the end dam at one end and slides on engineered bearings inside a longer support box at the other end.

Modular joints are classified based on the number of support bars at each interval and how they connect to the separation beams.

Picture 14 – Multiple support bar system.

Picture 15 – Single support bar system.

In a multiple support bar system, shown in Picture 14, each separation beam is supported on an individual support bar. The number of support bars equal the number of separation beams. The underside is congested and difficult to access for maintenance. In a single support bar system,

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all separation beams are supported by one large support bar at each interval as shown in Picture 15. It is more accessible for maintenance.

Based on past performance, the Ministry moved to single support bar modular expansion joints. In 2010, six modular expansion joints with multiple support bars on the QEW Niagara-bound Burlington Bay Skyway were replaced with single support bar modular joints. It was the first-time single support bar modular joints were installed on a Ministry project. The project, performance issues with the old joints, and staging challenges are described in [23].

Due to their complexity, modular expansion joints present difficulties for repairs. Their seals and equalizing springs can be replaced. However, there are limited options to repair cracked separation beams; weld repairs have proven ineffective and often fail shortly thereafter. Modular joints with field spliced separation beams, and especially older multi-support bar modular joints, are more prone to cracks in their separation beams. In older modular joints, their support beams are spaced further apart and used poor fatigue details at the shop splices as well as the welds between the support beams and the separation beams [23]. Currently, field splicing of modular joints is not permitted per *Structural Manual* Section 13.2.1.

P2-3.9.3 Design Requirements

P2-3.9.3.1 Installation of Joints, Before and After Paving

This section replaces Bridge Office Memorandum #2018-01, dated April 12, 2018.

Deck joint assemblies are currently required to be installed after the asphalt paving operation has been completed according to OPS Specifications. The method of installation generally involves creating a blockout cavity for the deck joint assembly, filling the cavity with Granular A, paving over the blockout, removing the asphalt and Granular A within the blockout limits, and installing the deck joint assembly. Installation after paving is beneficial since the asphalt is fully compacted up to the nosing angles, and the assemblies can be accurately placed very close to the final top of pavement elevations. This reduces dynamic loads applied to deck joint assemblies (increasing durability) and offers superior rideability compared to installing the assemblies prior to paving.

Installing deck joint assemblies after paving usually places the operation on the critical path of the construction schedule. Projects have the potential to be delayed and incur cost overruns if the designer and/or contractor does not fully account for this.

The Ministry often receives requests from contractors to install deck joint assemblies prior to paving. The Ministry's preference has historically been to install deck joint assemblies after paving, in accordance with the Contract Documents. It has been found that there are some instances where installation prior to paving is acceptable, such as when a reduced durability is acceptable over the remaining service life of the bridge. The designer would then need to clearly specify the accepted method of deck joint assembly installation in the Contract Documents.

The designer must specify the method of deck joint assembly installation in all bridge rehabilitation projects which require new deck joint assemblies as follows:

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1. The designer may choose to specify either installation prior to paving or after paving when all the following criteria are met:

- The bridge is not skewed.
- Average Annual Daily Traffic (AADT) is less than 2,000.
- The estimated working days for structure work, with installation prior to paving, is at least 10% less than with installation after paving.

The designer must specify installation after paving for all other scenarios unless written Approval is obtained by the designer from the Head of the Structural Section.

Installation prior to paving reduces the durability and longevity of the expansion joint. A service life of 20 years may be assumed for financial analysis.

P2-3.9.3.2 Proprietary Expansion Joint Seals

This section replaces Structures Office Memorandum #SCB-SO-2021-02, dated January 29, 2021.

Aside from the standard deck joint assemblies, there are other joints in a structure that must prevent water and debris infiltration while accommodating expansion, where standard Type A and Type C expansion joints are not necessary or suitable. Examples include:

- Vertical gaps between barrier walls on a deck and on adjacent wingwall where relative movement can occur.
- Longitudinal gaps between twinned structures.
- Shorter term holding strategies for transverse expansion joints on decks.

In 2021, the Ministry created a *Proprietary Joint Seal List* along with a new standard tender item and fill-in special provision (SP) to ensure these seals are specified in a consistent and competitive manner.

Proprietary expansion joint seals should not be used when a Type A or Type C expansion joint is required for standard installation and typical circumstances.

When a proprietary expansion joint is required, the designer must refer to the latest *Proprietary Joint Seal List,* available on MTO Technical Publications website. Proprietary seals are permissible only for the applications and locations shown in the list. Designers must include all joint seals from the list in the SP that suit the specified application, location, and movement.

The Ministry's Bearings and Expansion Joints Working Group is responsible for upkeeping the Proprietary Joint Seal List.

P2-3.9.4 OPS Specifications and References

OPS specifications, standard drawings, non-standard special provisions (NSSP) and contract design, estimating and documentation (CDED) chapters related to expansion joints are summarized in Table 30.

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Table 30 – OPS Specifications and References Related to Expansion Joints.

Description	OPS Specifications / Standard Drawing	NSSP (Custodian Offices)	CDED Chapter
Construction Specification for Deck Joint Assemblies, Preformed Seals, Joint Fillers, And Joint Sealing Compounds	OPSS.PROV 920	-	-
Material Specification For Deck Joint Assemblies	OPSS.PROV 1210	-	-
SSDs for Strip Seal Expansion Joints	SS113-10 to 18	-	-
SSDs for Modular Expansion Joints	SS113-20 to 22	-	-
Standard Drawings for Sliding Plate Expansion Joints	SS113-30 to 35	-	-
Standard Drawings for Expansion Joint and Sleeper Slab	SS113-36	-	-
Standard Drawings for Sequence of Expansion Joint Installation	SS113-38	-	-

A comprehensive repository of MTO technical documents can be found on the MTO Technical Publications website.

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