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**P2-3.10 Bearings**

Bearings transfer loads from the superstructure to substructure and accommodate rotation and movements of the superstructure. Elastomeric bearings and rotational (pot, disc and spherical) bearings are the two primary types of bearings used on MTO bridges.

Information pertaining to bearing replacement procedures, bridge jacking methods and potential remedial techniques for correcting inadequate bearing contact are described in the current version of the Ministry's *Bearing Replacement and Jacking Guidelines* [24]. This section is intended to be read in conjunction with those guidelines.

**P2-3.10.1 Background****P2-3.10.1.1 History of Bridge Bearings in Ontario**

The history of bridge bearings in Ontario provides valuable context for assessing the condition and performance of existing bearings.

Prior to 1960s, bridges generally relied on small sliding bearings consisting of steel, lead, and bronze plates. Rocker bearings were used for girder-type bridges and roller bearings were used for higher loaded girders. These bearings are functionally deficient, and most have been replaced.

In the 1960s, both plain elastomer pads and laminated elastomeric bearings were used. The laminated bearings were supplied by Andre Bearings and were individually moulded using natural rubber, and many remain in service today in good condition ([Picture 16](#) and [Picture 17](#)). Pot bearings were used for large loads, usually with the pot on the top and piston on the bottom. This is opposite to today's practice, with the pot on the bottom.



**Picture 16 – A laminated elastomeric bearing in service since 1965.**



**Picture 17 – Cross-section of a laminated elastomeric bearing from 1962.**

From 1970s to mid-1980s, laminated elastomeric bearings were cut from large sheets (like cutting a piece of cake in a sheet pan) and the exposed edges were wrapped with a layer of rubber. Over time, the cover layer often cracked or debonded, as illustrated in [Picture 18](#). Many of these

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bearings are now in poor condition. Neoprene, a type of synthetic rubber, was increasingly used for elastomeric bearings during this period.



**Picture 18 – Debonded cover of a laminated elastomeric bearing manufactured in the 1970s.**

Since then, bridge bearings have been designed in accordance with the requirements of the *OHBDC* and, more recently, the *CSA S6 CHBDC*. During this period, MTO also introduced the *Designated Sources of Materials (DSM)* program along with formal quality-assurance testing requirements for bearings. Laminated elastomeric bearings were vulcanized as a single, fully bonded unit and manufactured using natural rubber. Bearings produced under these standards are generally performing well and remain in good condition.

MTO has encountered polyurethane extrusion issues with certain disc bearings manufactured in the 1990s, however, the number of affected installations is relatively small.

#### **P2-3.10.1.2 Elastomeric bearings**

Elastomeric bearings are either plain or laminated. Plain bearings are up to 25 mm in thickness while laminated bearings typically range between 40 to 150 mm. [Picture 19](#) shows the cross-section of a laminated elastomeric bearing. Movements and rotation are accommodated through deformation and compression (refer to [Picture 20](#)) of the rubber. Elastomeric bearings are relatively low cost and are the most common type of bearings used on MTO bridges.

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Picture 19 – Cross-section of a laminated elastomeric bearing.



Picture 20 – An elastomeric bearing deforms to accommodate thermal movement.

### P2-3.10.1.3 Rotational Bearings

There are a few types of rotational bearings including pot, disc, and spherical bearings, with pot bearing being the most common type used on MTO bridges. Compared to elastomeric bearings, rotational bearings can support higher loads and accommodate larger movements. Rotational bearings can be fixed, unidirectional, or multidirectional. A unidirectional bearing typically has a guide bar along the centre, or on each side, to only allow horizontal movement in one direction (along the guide bars). Whereas a multidirectional bearing allows horizontal movement in both directions. Horizontal movements are accommodated by a low friction PTFE (Teflon) sliding interface which allows the superstructure to move relative to the bearing.

Pot, disc, and spherical bearings accommodate rotations differently. In a pot bearing, an elastomer is confined in a “pot” and behaves like a viscous fluid to accommodate rotation, refer to [Figure 16](#). A disassembled pot bearing is shown in [Picture 21](#).

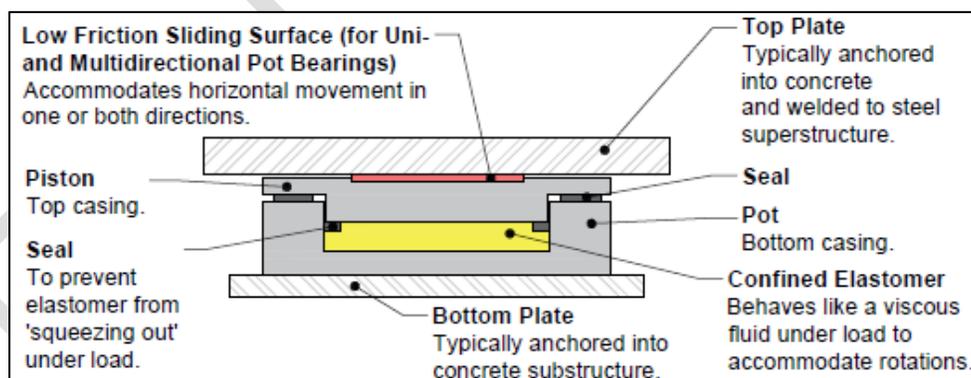
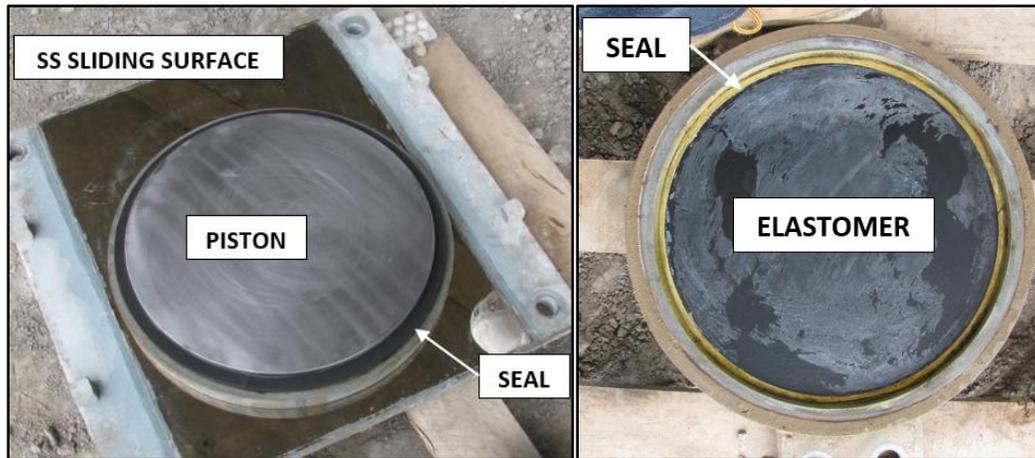


Figure 16 – An idealized cross-section of a pot bearing.

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Picture 21 – a disassembled pot bearing.

In a spherical bearing, there is a low friction sliding interface between the top convex and bottom concave plates. The plates move relative to each other to accommodate rotation, refer to [Figure 17](#).

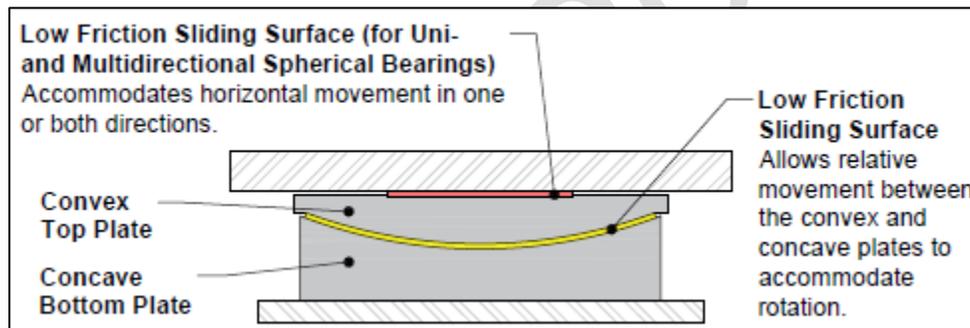


Figure 17 – An idealized cross-section of a spherical bearing.

In a disc bearing, a stiff urethane disc is sandwiched between the top and bottom plates, refer to [Figure 18](#), which deforms to accommodate rotation.

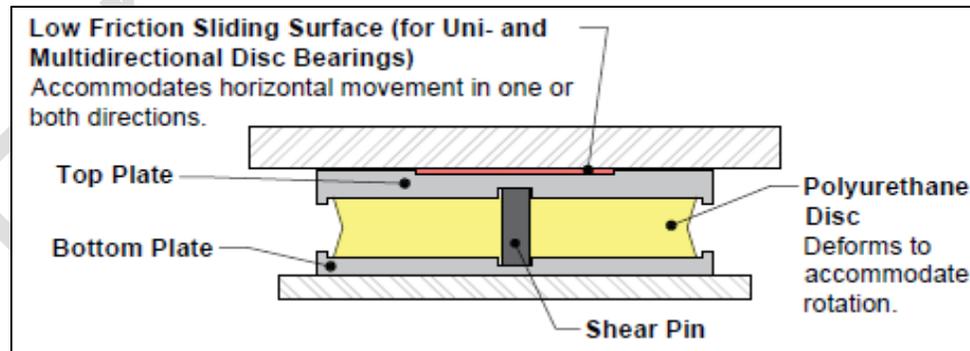


Figure 18 – An idealized cross-section of a disc bearing.

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**P2-3.10.2 Construction Details**

In general, bearings that can no longer accommodate the required rotations and movements, whether due to damage, misalignment, or changes in the structure's articulation, must be replaced, which in most cases will require jacking of the superstructure.

**P2-3.10.2.1 Repairing Elastomeric Bearings**

Maintenance of elastomeric bearings is generally not required unless the bearing has deteriorated to the point of failure, for example: excessive bulging, crushing, tearing or separation of laminations. When these conditions are present, the bearing must be replaced.

Although repair options for elastomeric bearings are limited, minor surface deficiencies can sometimes be addressed. Where fine surface cracking is present, but the bearing remains functional and otherwise in good condition, coating the surfaces with a compatible polymer-based sealant may be considered. When considering this type of repair, the designer must assess the remaining service life of the bearing and ensure that the selected intervention aligns with the anticipated service life of surrounding components.

**P2-3.10.2.2 Repair of Rotational Bearings**

PTFE sliding surfaces can be replaced when they become scored or damaged (refer to [Picture 22](#)). However, because replacement of the sliding elements requires jacking, seized rotational bearings are more commonly replaced in their entirety.



**Picture 22 – Unidirectional pot bearing where the sliding surface has rusted and peeled.**



**Picture 23 – Recoated pot bearing.**

Steel bearing assemblies exposed to excessive moisture and deicing runoff may be cleaned and coated (refer to [Picture 23](#)) to remove corrosion and maintain their performance.

In rare instances, the elastomer can squeeze out of a pot bearing (refer to [Picture 28](#)). The seal between the piston and pot should not be mistaken for the elastomer, (refer to [Picture 24](#)). The seal has no structural purpose; it is to prevent dust and water ingress. Broken seals can be removed, and a new backer rod can be installed into the gap for the same purpose.

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**Picture 24 – Seal between piston and pot is squeezed out.  
(Note: The pot for this bearing is at the top).**

### P2-3.10.3 Design Requirements

Bearing replacement involves jacking the superstructure onto temporary supports, removing existing bearings and repairing the surrounding elements, installing new bearings, and then lowering the superstructure. Jacking methods and replacement procedures are described in detail in the *Bearing Replacement and Jacking Guidelines*. Some points are reiterated here for emphasis. However, this section should be read in conjunction with those guidelines.

Reasons to replace existing bearings on a bridge may include:

- Bearings are in poor material condition.
  - Crushed or cracked elastomer ([Picture 25](#) and [Picture 26](#)), very severely corroded anchor rods, etc.



**Picture 25 – A crushed elastomeric bearing.**



**Picture 26 – Cracked elastomeric bearing with some rust staining.**

- Bearings are functionally obsolete and are no longer serving their intended purpose.
  - Excessive elastomer bulging.

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- Seized/tilted rocker bearings.
- Walking-out bearings ([Picture 27](#)).
- Elastomer squeezed out in rotational bearings ([Picture 28](#)).



**Picture 27 – An elastomeric bearing that has ‘walked out’.**



**Picture 28 – Elastomer (red) squeezed out in a disc bearing.**

- The articulation of a bridge has changed, requiring different performance criteria at some or all bearing locations.
  - Additional fixity provided for seismic performance or to change joint movement range.
  - Fixed location changed to accommodate translation.
  - Bearing fixity changed from fixed to expansion to accommodate flexible link slab.
- Design loads have increased since original design, and the current bearings no longer meet code requirements.
  - Semi-integral conversion, additional overlay, new design live load, etc.
  - However, bearings may still be acceptable if loads will not significantly increase, they are functioning well and show only minor material deficiencies.

Section 3.3.8 of the *Bearing Replacement and Jacking Guidelines* provides a summary of the advantages and disadvantages of different jacking methods, and recommendations on jacking locations and arrangements. In general, jacking from diaphragms or directly from girders are preferred. In some cases, it is necessary to jack from pockets in substructure or from temporary supports resting on the existing footing. Jacking from lifting tabs or a strong back system requires additional considerations. Jacking from corbels anchored to substructure is the least preferred option.

### **P2-3.10.3.1 Design for Full Contact**

Bearings that do not achieve full contact with the surfaces above and/or below them can experience reduced durability, accelerated deterioration and poor performance. Appendix A in the *Bearing Replacement and Jacking Guidelines* highlights remedial measures for situations where full contact is not achieved during construction.

For elastomeric bearings, however, injecting polymer materials into gaps has shown inconsistent performance and is sensitive to environmental conditions. Existing bearing contact issues shall

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be addressed by reconstructing contact planes with the proper use of the template method. Polymer injection is not permitted in rehabilitation design.

Where existing steel girder flanges exhibit warping, titanium putty has been used to fill in gaps to provide contact with shoe plates. Filler plates may also be specified but shall be kept to a minimum and shall be fixed in place. Filler plates and shoe plates may require machined bevels to match the existing geometry.

#### **P2-3.10.4 OPS Specifications and References**

Relevant OPS specifications, non-standard special provisions (NSSP) and contract design, estimating and documentation (CDED) chapters related to bearing repair and replacements are summarized in [Table 31](#).

**Table 31 – OPS Specifications and References for Bearings.**

<b>Means and Methods</b>	<b>OPS Specifications</b>	<b>NSSP (Custodian Offices)</b>	<b>CDED Chapter</b>
Installation of Bearings	OPSS.PROV 922	-	B922
Jacking	OPSS.PROV 921	-	B921
Bearing Repairs	-	(1)	-

<sup>1</sup> NSSPs for bearing repairs are generally project-specific, based on the scope of repairs.

A comprehensive repository of MTO technical documents can be found on the [MTO Technical Publications](#) website.

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**P2-3.11 Waterproofing****P2-3.11.1 Background**

Waterproofing systems provide an impermeable barrier to protect concrete and its reinforcing steel from damages due to chloride ion penetration. There are many different types of waterproofing systems, including sealers, spray-applied polymer membranes, hot-applied (poured) rubberized asphalt membranes, torch-applied and self-adhering rubberized asphalt sheet membranes etc. [25]. Typically, the waterproofing systems are directly applied to the concrete surface and then topped with asphalt.

Since 1978, it has been part of the Ministry's corrosion protection policy to waterproof bridge decks using a hot-applied rubberized asphalt membrane, protected with asphalt impregnated protection boards, and topped with two 40 mm lifts asphalt. Refer to Section 1.3.2 in Part 1 of the 2007 *Structural Rehabilitation Manual* for a comprehensive timeline of Ministry's bridge deck waterproofing practice.

Due consideration shall be given to waterproofing of buried structures taking into account the depth of fill, risk of leakage and deterioration, access, and associated highway works. When feasible, waterproofing of concrete culverts shall be in accordance with *Structural Manual* and soil-metal structures shall receive an impervious membrane placed in the backfill. Further information may be found in *CSA S6.1 CHBDC Commentary*.

**P2-3.11.2 Construction Details**

When the concrete substrate temperature increases during waterproofing application, expanding air and water vapour trapped in surface pores can cause membrane blistering, [25]. To minimize this risk, concrete surfaces must be clean and dry before application. It is best practice to install waterproofing when deck temperatures are stable or decreasing. Decks with significant surface voids are particularly prone to blistering. As part of the November 2023 update to *OPSS.PROV 914*, the installation requirements were revised, where the waterproofing membrane is applied in two layers and interlayered with membrane reinforcement, to reduce the risks of blistering.

Achieving consistent and adequate membrane thickness requires proper surface preparation; excessively rough or textured deck surfaces must be ground or overlaid.

To install hot-applied rubberized asphalt membrane, cakes of asphalt waterproofing membrane are melted on site in a mechanically agitated heating and mixing kettle. A layer of tack coat is first applied to the prepared concrete surface. The melted asphalt membrane is then squeegeed onto the deck surface. Asphalt impregnated protection boards are laid on top of the membrane while the surface is still tacky. Finally, the Ministry's standard for paving bridge decks is two 40 mm lifts of asphalt.

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**P2-3.11.3 Design Requirements**

To ensure proper adhesion and performance, *OPSS.PROV 914* requires a minimum air and concrete surface temperature of 5°C when applying bridge deck waterproofing. As waterproofing provides the critical protection measure for concrete decks, rehabilitation work shall be planned conservatively to permit the waterproofing to occur during favourable seasonal temperatures for the site.

Waterproofing shall be removed and replaced as part of a bridge rehabilitation. It is recommended to avoid partially removing and replacing waterproofing, as it is very difficult to achieve a clean and proper lap between existing and new membranes, refer to [Picture 29](#).



**Picture 29 – Difficulty lapping new waterproofing with existing.**

Where the work is staged, the waterproofing membrane, protection boards and asphalt must be staggered between stages to offset their construction joints, refer to [Figure 19](#). *OPSS.PROV 914* specifies a minimum 150 mm lap. Achieving the minimum lap width is not always feasible due to bridge geometry and staging requirements. Nevertheless, it should be maximized and ensure membrane, protection board and asphalt joints do not stack on top of each other.

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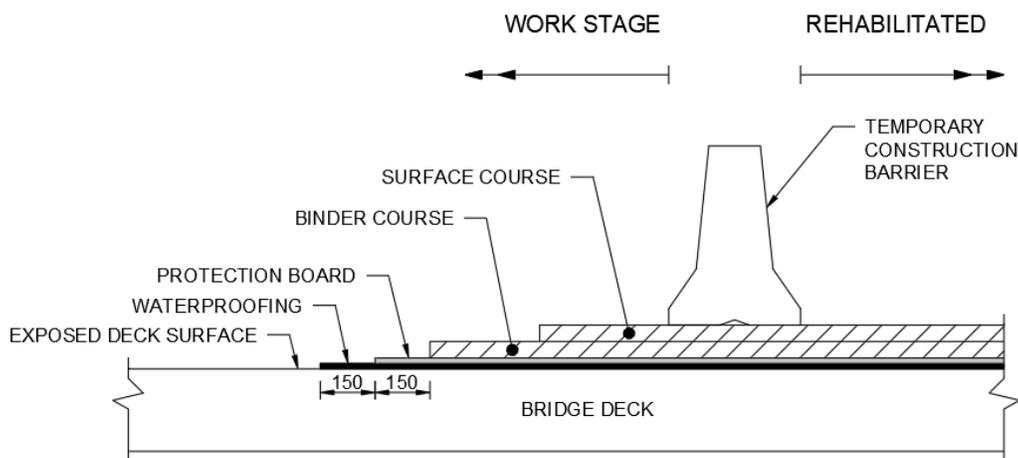


Figure 19 – Typical waterproofing longitudinal joint detail for staged construction.

For carryover construction, the waterproofing membrane and protection board laps are typically protected by a layer of bond breaker with blocking and are temporarily paved over during winter shutdown (refer to [Figure 20](#)).

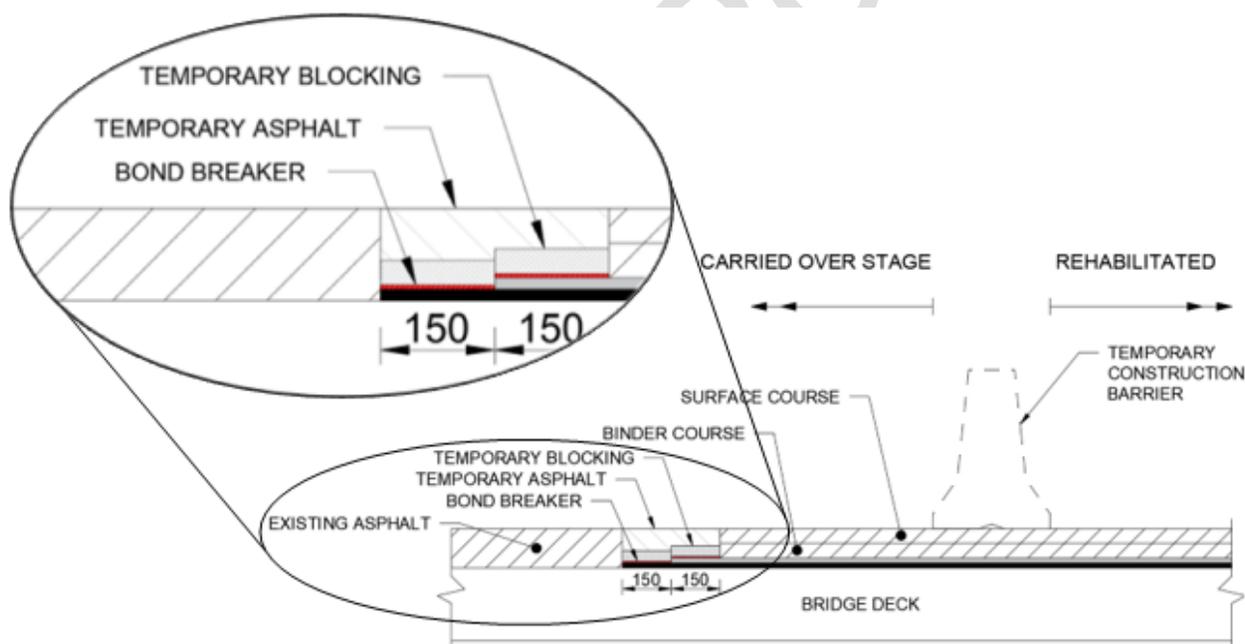


Figure 20 – Typical waterproofing longitudinal joint detail for carryover construction.

#### P2-3.11.4 OPS Specifications and References

Relevant OPS specifications, non-standard special provisions (NSSP) and contract design, estimating and documentation (CDED) chapters related to bridge deck waterproofing are summarized in [Table 32](#).

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**Table 32 – OPS Specifications and References for Bridge Deck Waterproofing.**

Means and Methods	OPS Specifications	NSSP (Custodian Offices)	CDED Chapter
Hot Applied Asphalt Membrane	OPSS.PROV 914	-	-

A comprehensive repository of MTO technical documents can be found on the [MTO Technical Publications](#) website.

For TCP Posting

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### P2-3.12 Repairs and Strengthening using Fibre Reinforced Polymers (FRP)

#### P2-3.12.1 Background

Fibre reinforced polymers (FRP) are composite materials consisting of fibres that are bonded together with a resin matrix. FRP sheets, plates, and bars are used in various systems. These FRP systems have been used on Ministry projects for strengthening and to improve the durability of concrete components. In some jurisdictions, FRP systems have been used to rehabilitate steel components.

FRP systems used in rehabilitations are generally categorized as hand applied wet lay-up systems, or bonded prefabricated systems [26].

FRP is lightweight, can conform to the geometry of a member, and can be applied overhead with negligible reduction of vertical clearance. FRP systems generally require less access and are faster to install compared to other rehabilitation methods using concrete.

#### P2-3.12.2 Construction Details

The installation of FRP systems must be according to the Fibre-Reinforced Structures section of the *CSA S6 CHBDC*, which describes the wet layup process as impregnation of the reinforcing fibres with a saturating resin in a manner recommended by the FRP manufacturer to obtain full saturation. This is often accomplished through a process where the FRP sheets are first soaked in a resin bath, and the saturated sheets are then placed onto the prepared concrete surface while still 'wet'. In some cases, manufacturers permit an alternative procedure in which the substrate is first coated with resin, dry FRP sheets are applied and rolled into the wet resin, and additional resin coats are then applied and worked into the sheets until full impregnation is achieved.

As the resin cures, it hardens and adheres to the concrete. Upon curing, a compatible paint coating is applied to protect the FRP repairs against degradation due to ultraviolet radiation.

Bonded prefabricated systems utilize prefabricated FRP plates that have already been cured before installation. These plates are adhered to the concrete surface with an adhesive.

Tensile bond tests are specified in the Ministry's construction specification to ensure adequate impregnation and bond are developed.

#### P2-3.12.3 Design Requirements

*CSA S6 CHBDC* permits the use of aramid (AFRP), glass (GFRP), and carbon (CFRP) fibres. GFRP and CFRP have been used on Ministry projects, with CFRP being more common. The material cost of FRP fabric is small compared to the costs of access, labour, and equipment. The Ministry requires minimum of two layers for FRP for wrapping applications. Due to its higher material strength, it is preferable to use fewer layers of CFRP than more layers of GFRP.

Applications of FRP are categorized as bond-critical or contact-critical:

**Bond-critical applications** rely on the bond to the substrate for load transfer, such as shear or flexural strengthening.

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**Contact-critical applications** rely on contact between the FRP and substrate, such as wrapping a column to provide confinement.

*CSA S6-25 CHBDC* requires the concrete surface to have a minimum tensile strength of 1.5 MPa for bond-critical applications. This needs to be confirmed during the design phase where a bond-critical application of FRP is being specified.

FRP can be used to strengthen a member to provide additional shear or flexure capacities. It can also be used to improve torsional capacity, though this application is less common.

Proprietary FRP anchorages are not permitted unless test results can demonstrate full development to the satisfaction of the owner.

### **P2-3.12.3.1 Strengthening using FRP**

Shear strengthening using FRP typically involves externally bonding FRP strips to the deficient member. This method has been used to strengthen shear-deficient girders on Ministry projects. The FRP strips act as additional external stirrups. While it is most desirable to fully wrap around an entire cross-section (like a closed stirrup), it is not practical where the deck is integral with the girders. Instead, the FRP strips are generally U-shaped, and anchored in the compression zone with additional horizontal strips, refer to [Picture 30](#). *CSA S6 CHBDC* provisions for calculating FRP shear resistance are developed for beams with rectangular or T-sections. For components with other cross-sections, a more rigorous analysis is required. Fibre direction and installation sequence of FRP layers shall be clearly indicated on the drawings.

Flexural strengthening using FRP involves externally bonded FRP sheets or plates to the tension face, increasing the flexural capacity. Unlike steel, FRP exhibits a lower elastic modulus and a linear elastic response up to failure. Under the same load, FRP will experience greater deformation compared to steel. Consequently, to fully mobilize the FRP's tensile capacity, the strengthened element must undergo larger deformations than an element retrofitted with steel plate or reinforcement. Strengthening within an envelope encompassing 6 m above the travelled lanes shall consider the requirements described in Section [P2-3.12.3.4 P1-1](#) Impact Repairs.



**Picture 30 – Shear strengthening using FRP.**

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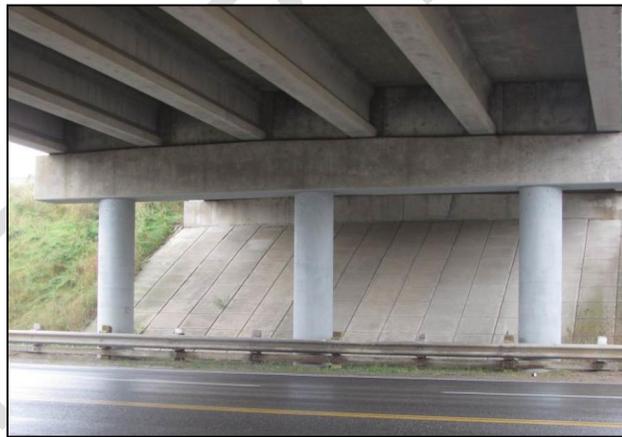
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**P2-3.12.3.2 Wrapping Circular Columns with FRP**

This method involves externally bonding FRP sheets to circular concrete columns. The FRP wrapping confines the concrete and improves its capacity and ductility without adding significant stiffness or mass to the column. FRP wrapping is not recommended for non-circular columns where the lateral stresses are not uniform and thus reduces the effectiveness of confinement.

While the Ministry has used FRP wrapping systems to strengthen columns, it is more commonly used as a durability-improving measure to complement patch repairs. It is difficult to access and remove concrete from behind closely spaced spirals, and doing so often requires staging or temporary supports. Instead, delaminated concrete is generally removed to the mid depth of the spiral reinforcing steel. It is preferable to remove chloride contaminated concrete prior to wrapping as expansive corrosive forces may fail the wrap. When chloride contaminated concrete is not completely removed, the column will continue to undergo corrosion, however FRP wrapping does provide an impermeable barrier that seals out moisture, potentially slowing down the corrosion activity. The degree of contamination and appropriateness of FRP wrapping shall be determined by the designer.

Wrapped pier columns are shown in [Picture 31](#). Wraps should extend the full height of the columns. For durability improvements it is not necessary to wrap a column whenever it is patched but it can be considered for columns at risk of macro cell corrosion with extensive patching, and where aesthetics is more critical. All columns within a support group should be wrapped together for consistent appearance and protection.



**Picture 31 – Columns wrapped with FRP.**

**P2-3.12.3.3 Durability Repairs using FRP**

FRP have been used to wrap repaired soffit and girder end patches, where it is not relied upon for additional capacity but as a durability measure to prevent the patches from debonding. Designers shall consider the impact of remaining chloride contaminated concrete as described in the previous section.

For soffits with low concrete cover, it is not always desirable to build-out patches as they are unsightly and may encroach on the vertical clearance. Instead, an alternate strategy is to complete shallow patch repairs followed by wrapping with FRP. This improves durability of the

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shallow patches while achieving a uniform surface that blends with the existing soffit. [Picture 32](#) shows patches on the bottom flanges of concrete trapezoidal box girders that are wrapped with FRP. Similarly, FRP used to wrap girder end patches is shown in [Picture 33](#).



**Picture 32 – Patches on bottom flanges of concrete trapezoidal box girders wrapped with FRP.**



**Picture 33 – Girder ends wrapped with FRP.**

#### **P2-3.12.3.4 Impact Repairs**

In some cases, FRP has been used to wrap patches that were initially damaged by an over height vehicle/load strike. If a structure has low clearance and a history of load strikes, wrapping the impacted areas with FRP should be avoided. The wrap is likely to be damaged again, as illustrated in [Picture 34](#), and there are limited options to repairing them.



**Picture 34 – Damaged FRP wrap from subsequent load strike.**

#### **P2-3.12.4 OPS Specifications and References**

Relevant OPS specifications, non-standard special provisions (NSSP) and contract design, estimating and documentation (CDED) chapters related to fibre-reinforced polymer (FRP) systems are summarized in [Table 33](#).

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**Table 33 – OPS Specifications and References for Fibre-Reinforced Polymer (FRP) Systems.**

<b>Means and Methods</b>	<b>OPS Specifications</b>	<b>NSSP (Custodian Offices)</b>	<b>CDED Chapter</b>
Fibre-Reinforced Polymer Wrapping	-	Yes (Structures Office)	-
Fibre-Reinforced Polymer Jacket for Pile Repair and Protection	-	Yes (Structures Office & Foundations Office)	-

A comprehensive repository of MTO technical documents can be found on the [MTO Technical Publications](#) website.

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**P2-3.13 Repairs to Prestressed & Post-Tensioned Components****P2-3.13.1 Background**

Prestressed components require special consideration when undertaking repairs. Concrete removals reduce the effective prestress area and increase the compression on the remaining section. Impacts to secondary prestress shall be accounted for, and staged removals shall always be considered when undertaking removals of prestressed components. In addition to removals, any resulting concrete repairs result in areas lacking precompression. These areas shall be protected with supplemental measures to match the service life of the prestressed components.

CSA S6 does not permit designers to account for interface shear transfer between normal concrete and prestressed concrete unless mechanical anchorage is provided.

In special cases, designers have specified preloading of the structure in an attempt to elicit compression into patch repairs through elastic response of the structure. In most cases, this is considered impractical and should be avoided. Load sharing and stiff composite sections including increased stiffness contributions from barriers, curbs and sidewalks all limit the effectiveness of this approach.

**P2-3.13.2 Girder End Repairs**

Bridges with expansion joints commonly exhibit concrete deterioration at their girder ends, particularly on the bottom flange. The application of elastomeric sealants and grout parging at the ends of strands have not been effective at preventing the long-term ingress of moisture along strands resulting in corrosion. More recently, bridges with retrofitted semi-integral abutments have shown similar deterioration years after being converted from deck joints. This arises from lasting contamination by de-icing chemicals that leaked through the joints, causing damage to propagate many years later, despite being undetected at the time of conversion.

A common practice for girder end repair has consisted of shallow surface repairs on deteriorated prestressed girder ends. These repairs typically involve patching the concrete cover rather than undertaking more in-depth removal and repairs, which would remove additional chloride-contaminated concrete. Surface repairs do not address the underlying source of the deterioration and lead to cracking and delamination of the new shallow patches.

Designers should avoid shallow surface repairs at girder ends and instead remove chloride-contaminated concrete where feasible, supplementing with measures such as encapsulation or cathodic protection (refer to Section [P2-3.8](#) Cathodic Protection) when deeper removal is structurally impractical, to ensure durable long-term performance.

Alternatively, bulk encapsulation of the deteriorated girder ends provide additional confinement to the girder ends and simplifies the concrete placement operation and shown in [Picture 35](#).

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**Picture 35 – Encapsulated prestressed girder ends.**

In general, it is beneficial to remove as much chloride-contaminated concrete as structurally feasible. However, removing concrete around existing prestressing strands at girder ends may not be acceptable to meet strength and serviceability limit states. Feasibility of concrete removal beyond the cover concrete must be analyzed for local effects, considering strand development, shear and moment capacity, end D-region forces due to the support reaction and prestress forces. The corresponding short- and long-term impacts must be considered.

#### **P2-3.13.3     Repairing Impact Damages**

Repairing impact damages to prestressed components requires a thorough examination of the defects, including any cracking, damages to mild steel reinforcing, particularly around the anchorage of stirrups, and assessment of any damage to prestressing tendons. Once the extent of damage is confirmed, concrete repairs are typically carried out through form and pump with either normal concrete or self-consolidating concrete.

#### **P2-3.13.4     Repairing Broken Strands**

Repairing prestressing strands in precast girders is typically limited to splicing strands using proprietary couplers to restore continuity or reintroduce prestress locally in a debonded repair zone. Full-scale tensile proof tests should be carried out to verify the capacity of the coupler system, especially when limited data is available. Couplers require sufficient space for installation and usually need to be offset to accommodate typical strand spacing.

The extent of capacity loss from strand and wire breakage should be considered together with the intended service life of the overall rehabilitation strategy. In some cases, it may be acceptable to leave broken strands unrepaired, or an alternative method of strengthening may be used to supplement ultimate limit state capacity where a significant portion of strands have been damaged.

#### **P2-3.13.5     Repairing Unfilled Post-Tensioned Ducts**

Voids in post-tensioned ducts can occur due to incomplete filling, leakage, or bleeding, where excess water rises through unfilled interstitial spaces between wires and strands and is reabsorbed

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into the grout, resulting in a void. These voids are most common at the high points along the tendon profile. Impact echo (refer to Section [P1-11.2 Impact Echo and Pulse Velocity](#)) and other non-destructive testing methods can be used to identify suspected voids. Follow-up physical testing, such as a borescope inspection or open 'test pits', are generally required to confirm the extent of voids and condition of existing grout. Sampling and testing of grout are also necessary to confirm its properties such as chloride content, pH, carbonation, which will influence the need for and extent of repairs.

Repairs may involve filling voids with new grouts, or impregnation of the strands through the interstitial spaces to form a protective coating.

**P2-3.13.6 Removing Concrete around Post-Tensioned Anchorage Zones**

Removals in post-tensioned anchor zones should be limited to approximately the mid-depth of the bursting steel and clearly indicated on the drawings. Removals around spalling reinforcement should account for locked-in tensile stresses. Additional care in detailing is required when removing concrete behind the mild reinforcing steel at post-tensioned anchorages.

**P2-3.13.7 Concrete Removal and Repairs around Voids**

Care must be taken when identifying deteriorated concrete around voids, particularly on the soffit. Attempts to locate the voids shall be made during the pre-engineering investigation stage as described in Section [P1-10.1.2 GPR Assessment of Other Components](#). Provisions should be specified in the contract for the accidental puncture of voids and the installation of lost forms as appropriate.

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**P2-3.14 Repairs to Structural Steel****P2-3.14.1 Background**

Since the 1960s, MTO has moved towards atmospheric corrosion resistant (ACR) steel to reduce long-term maintenance costs of steel bridges. Atmospheric corrosion resistant (ACR) steel is low-alloy mild steel, most commonly associated in the literature with the trade name 'Cor-Ten' and has been available since roughly 1933. The history of ACR steel is recorded elsewhere, but relevant dates for MTO bridge rehabilitation include:

- *ASTM A242* published in 1941 (approximately Cor-Ten A).
- *ASTM A588* published in 1964 (approximately Cor-Ten B).
- *ASTM A709* approved in 1974.

A242 steel has typically been associated with architectural applications, and it is not expected that any A242 steel remains in service in the MTO bridge inventory.

MTO specifications have mandated the use of *CSA G40.20/G40.21* Type A or AT steel but *ASTM A588* steels designated "W" for enhanced atmospheric corrosion resistance have generally been considered equivalent. Availability of plates has often resulted in *ASTM A588* steel being supplied for new girder fabrications.

*ASTM A709* was approved in 1974 and consolidated several steel specifications into a single specification. High performance steels (HPS) were introduced in *ASTM A709*. HPS 50W may be encountered but is not generally expected. Higher yield grades (HPS 70W, HPS 100W) have not been found to be cost effective.

The Canadian Institute of Steel Construction (CISC) provides [technical resources](#) on historical listing of obsolete Canadian structural steel grades. The American Institute of Steel Construction (AISC) *Design Guide 15: Rehabilitation and Retrofit* and *Design Guide 21: Welded Connections – A Primer for Engineers* also contain useful information on historic steel specifications.

**P2-3.14.2 Construction Details****P2-3.14.2.1 Weldability**

For bridges built with historic structural steel designations where the chemical composition requirements are uncertain, it is prudent to assess its weldability. Chemical composition can be determined in accordance with Section [P1-6](#) Requirements for Structural Steel Investigations. An off-structure weld test should be conducted if material properties cannot be confirmed before welding, if steels are available, such as from a component that has been removed and replaced.

Most steels are weldable, but the ease of welding, or their weldability, can vary considerably. The chemical composition, particularly carbon content has a significant impact on the weldability of steel. *CSA G40.21* specifies the chemical composition requirements for modern grades of structural steel. For example, the carbon content for 350A and 350AT grade structural steels is limited to a maximum of 0.20%.

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ASTM A7 is an example of a historic structural steel designation, it was first introduced in the early 1900s and became obsolete in 1967. The specification did not control carbon content and mill practices evolved over that time. As such, the weldability of A7 structural steel must be evaluated on an individual basis.

### P2-3.14.2.2 Non-Destructive Testing for Steel and Welds

*OPSS.PROV 906* includes requirements for magnetic particle (MT), ultrasonic (UT), and radiographic (RT) testing, including qualifications of the testing personnel and firm. Inspection requirements in *OPSS.PROV 906*, such as testing frequency, are geared towards new structural steel fabrication. For rehabilitation projects involving repairs to existing structural steel, those requirements may need to be modified based on project-specific needs and conditions.

Magnetic particle testing (MT) reveals surface and near-surface discontinuities by applying a magnetic field, typically induced by a handheld yoke, to the object. The tested material must be ferromagnetic, such as carbon steel or martensitic stainless steel. Discontinuities are observed by how they impact the magnetic field. Fine iron particles are dusted onto the surface, accumulating at the discontinuities to make them visible. Thin, nonconductive, coatings will normally not interfere with the formation of indications. However, for coating with a thickness greater than 2 mil, it must be demonstrated using a test block, that unacceptable discontinuities can be detected through the maximum thickness of coating applied to the component to be tested.

Liquid penetrant testing (PT) involves applying a penetrant to the surface of a material, which seeps into any discontinuities through capillary action. For most field inspections, a solvent removable, visible dye is used as the penetrant, with a non-aqueous developer to both draw out the penetrant and provide contrast. Liquid penetrant testing can only be used to detect open-to-surface discontinuities. Unlike magnetic particle testing, liquid penetrant testing can be used on non-ferrous material, such as aluminum and austenitic stainless steel. However, it is more sensitive to surface contamination and can only be used on non-porous surfaces.

Ultrasonic testing (UT) uses a piezoelectric transducer to both generate and detect ultrasonic waves. By analyzing the characteristics of the reflected wave, ultrasonic testing can be used to locate internal defects in steel. It is effective in detecting internal discontinuities such as cracks, lack of fusion, lack of penetration, porosity, etc. Ultrasonic testing can also be used to measure thicknesses where the section is only accessible from one side.

Radiographic testing (RT) works similarly to a medical x-ray but typically uses gamma rays for inspection. A source of ionizing radiation, such as isotopes of iridium or cobalt, is positioned on one side of the steel, and a radiation-sensitive medium, typically radiographic film, is placed on the opposite side. The emitted radiation passes through the steel and exposes the medium. The radiograph, akin to photographic negatives, is then examined to identify internal defects within the steel. Radiographic testing is generally not used for rehabilitation projects due to the additional safety precautions required due to radiation.

*FHWA-HIF-19-088 Bridge Welding Reference Manual* [27] and *CSA W59* describe these, as well as other, non-destructive testing methods in greater detail.

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### P2-3.14.2.3 Detailing Considerations

#### Preinstallation Verification for Turn-of-Nut Bolt Installations

In new construction and rehabilitation projects involving substantial structural steel work, the work is typically carried out by sub-contractors who specialize in ironwork. For smaller rehabilitation projects with limited structural steel scope, the contractor may have less experience working with structural steel.

Pre-installation verification requirements are added to the July 2025 version of *OPSS.PROV.906*, similar to the requirements from the *Research Council on Structural Connections (RCSC) Specification for Structural Joints Using High Strength Bolts* to demonstrate proficiency with the 'turn-of-nut' installation method before commencing repairs. Particularly for smaller projects involving limited bolt installation, pre-installation verification helps confirm installer competency, proper understanding of the installation procedure, and the ability to achieve the required bolt tension, thereby reducing the risk of improper installation and subsequent performance issues.

#### Mock-Ups

For extensive and repetitive repairs, requirement for a mock-up should be included. The mock-up should replicate the existing in-situ conditions, and repairs should be performed on the mock-up and approved before being carried out on the bridge. The mock-up can serve as a reference for accepting repairs on the bridge. Repairs on the bridge should be performed by the same workers who complete the repairs on the mock-up.

#### Holes

Rehabilitation work often requires field drilling holes to retrofit new steel or create access points (e.g., rat holes) for repairs. According to *OPSS.PROV 906*, these field-drilled holes must be drilled or sub-drilled and then reamed. Reaming removes burrs and irregularities from the drilling process, providing a smooth surface finish and helping to prevent stress concentrations. Additionally, rat holes shall be sealed after completion of the work, for example using plastic filler caps. Rat holes shall not be welded closed. Sealing is required to prevent the accumulation of debris and water within crevices and inside box girders.

### P2-3.14.3 Design Requirements

#### P2-3.14.3.1 Fatigue Retrofits

MTO carries out periodic fatigue inspection to identify fatigue cracks, poor fatigue details and estimate the remaining fatigue life. Evaluations often show that the fatigue life has been exhausted even in the absence of visible fatigue cracks.

Rehabilitating fatigue cracks and fatigue-prone details involves both preventative and reactive strategies. Key considerations include whether the member is a primary or secondary member, the orientation of cracks and the system's redundancy. For trusses, two girder bridges, other non-redundant steel bridges, and well-known fatigue-prone details such as partially welded cover

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plates, proactive retrofits should be considered. In other cases, a balanced approach that involves regular fatigue inspection and risk assessment may be appropriate. Structural monitoring and instrumentation can refine fatigue evaluations by collecting precise data on service stresses and strains.

### P2-3.14.3.2 Distortion-induced Fatigue

Distortion-induced fatigue should be addressed relative to traffic volume on the bridge. For example, MTO has observed, through many similar underpasses over Highway 400, that the occurrence of cracking relates directly to the traffic volume on the bridge and the stress range. *CSA S6 CHBDC* conservatively assumes that all trucks contribute to fatigue damage; however, research has shown that a more refined assessment can be made by using load-test data that records stresses at the bottom of the web from passing trucks to estimate the proportion of truck traffic that actually produces fatigue-inducing effects [28].

Traffic volume and percentage of truck vehicles significantly influence the urgency of the retrofit work. Considerations should be given for grouping and completing similar repairs across several bridges together or coordinating with other major preventative maintenance work on the bridge when traffic control and access is already in place.

Most distortion-induced fatigue cracks in bridge girders are caused by differential deflections between adjacent girders resulting in secondary forces transferred through the floor beams, diaphragms, or cross frames.

Diaphragms, cross frames, lateral bracing, and floor beams are generally connected to the girder web using a vertical stiffener (e.g., connection plate), as shown in [Picture 36](#). Considerations for connecting the girder flanges and the connection plate were first mentioned in the 1991 edition of the *OHBDC*. However, it was not explicitly specified until the adoption of *CSA S6-00 CHBDC* which required connection plates to be welded or bolted to both flanges of the girder.

For bridges constructed prior to the 1990s, it was common practice to cut the vertical stiffener short, leaving a 20 to 50 mm gap between the end of the stiffener and the flange. Often, the fillet welds attaching the stiffener to the web were also terminated short of the stiffener end, resulting in an even larger gap, as shown in [Picture 36 \(Detail 1\)](#).

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**Picture 36 – Typical internal bracing connection detail of box girder bridges constructed prior to 1990.**

If lateral movement of the top flange relative to the bottom flange takes place, large strains are imposed in the gap region because of the significant change in stiffness between regions of the web. Because of this, this phenomenon is sometimes described as ‘web gap distortion’. Distortion-induced fatigue is especially prevalent on bridges designed prior to the 1990s.

In Ontario, cracks were first reported in steel box girders in the late 1990s at bottom of cut-short vertical stiffeners connected to intermediate cross bracing located in the positive moment region.

Cracks are typically addressed through hole drilling. However, hole drilling alone often isn’t sufficient without addressing the source of the problem. Several treatments are suitable to correct the web-gap issues. Chapter 4 of *Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges* [29] provides a few different options to positively attach the cut-short vertical stiffener to the flange, including using welded, bolted or hybrid connections. It is advisable to avoid or minimize field welding. Bolting the stiffeners to the flange with structural steel ‘clip angles’ is a common retrofit, and similar details have been successfully used on many MTO bridges. This method can readily be undertaken in the field with familiar equipment.

### P2-3.14.3.3 Post-Weld Fatigue Treatments

Annex R of CSA W59 outlines several post-weld treatments to improve fatigue performance of welds, including toe grinding, hammer peening, ultrasonic impact technology (UIT) and other techniques.

Toe grinding is a mechanical process that smooths out sharp weld profiles to reduce stress concentrations. The process is carried out using a grinder with a tungsten carbide burr and its tip radius varies depending on the plate thickness.

Hammer peening involves mechanically impacting the surface of a weld. UIT works similarly to hammer peening, but impact is delivered through high-frequency vibrations. Both hammer

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peening and UIT introduce plastic deformation to the weld to improve the weld toe geometry and produce compressive residual stresses. The main difference is the equipment used, UIT impacts at a much higher rate than hammer peening, the application can generally be done quicker. UIT equipment is more precise and repeatable and can be easier for the operator to use.

There are several other post-weld treatments available, however, toe grinding, hammer peening and UIT are the most used treatments on MTO projects.

### P2-3.14.3.4 Repairing Impact Damages

#### Heat Straightening

Heat straightening is a repair technique that uses controlled heating and cooling to restore the metal back to its original shape. It is used to correct deformations or distortions in steel, typically after a load strike. Heat straightening was first used by the Ministry in 2012 at the Highway 417 Lees Avenue Underpass in Ottawa to repair the exterior girder which was damaged by an over-height vehicle. The strike caused a 60 mm long crack extending through the web and completely across the bottom flange. The resulting misalignment was about 50 mm on either side of the crack along the bottom flange. The girder was successfully restored to its original geometry using heat straightening, and new plates were bolted to its web and bottom flange at the fractured location. The Ministry has used heat straightening on other projects since, with mixed results. The procedure for heat straightening – including the heating pattern, sequence, and temperature – depends on the steel properties and the type, location, and extent of damage. It must be designed by a qualified engineer with experience in heat straightening and requires coordination with the contractor on site to ensure that both the procedures are followed and that the steel reacts as intended to the heating. Heat straightening differs from other heat forming repairs such as hot mechanical straightening or forging, in that it primarily relies on heat rather than external applied force to deform the steel.

This technique is extensively covered in the *FHWA-HIF-23-003 Manual for Heat Straightening, Heat Curving and Cold Bending of Bridge Components* [30].

#### Repairing Gouges and Notches

There are cases where existing structural steel, such as the top flange of girders, is damaged by concrete removal equipment during rehabilitation, causing gouges and notches. These sharp irregularities on the surface create localized areas of stress concentration and can cause cracks to develop. Repairs typically involve grinding to smooth out and ensure a uniform surface to reduce stress concentration. The ground area is tapered, generally 1V:10H, to provide a gradual transition. Grinding marks shall be parallel to the direction of rolling, in the same direction as the steel's grain. For deep gouges, grinding alone could remove significant amount of material due to the taper, making it impractical or structurally unfeasible. In such cases, weld repairs, according to the requirements of CSA W59, may be required.

Load strikes often cause punctures, gouges, and notches. In the 2000s, four box girder underpasses over Highway 402 were struck, with repairs varying according to the severity of the damage - from notch and weld repairs to more extensive retrofits. Research [31] evaluated the effectiveness of these repairs using photogrammetry to capture the impact deformation geometry, along with finite element modelling validated against load test data. They found that impact

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damage creates complex geometries with high stress gradients and secondary bending stresses, which are highly sensitive to small changes in deformation and geometry. In some cases, the as-deformed condition can still meet fatigue limit state requirements. In more severe instances, bolted gussets can be used to complement the as-deformed geometry and control secondary bending stresses at the impact area.

### P2-3.14.3.5 Repairing Deteriorated ACR Steel Patina

Like many other North American jurisdictions, the MTO has experienced mixed performance of ACR steels. Through observation, some rapidly deteriorating problem areas (e.g., under expansion joints, at splices, etc.) were identified for proactive coating, and minimum clearance requirements above travelled lanes were introduced in the *Structural Manual*.

Some ACR structures are performing very well, while others have continued to experience deterioration issues. In 1989, FHWA reported on problematic development of the protective patina on structures with low clearance to water bodies, requiring minimum clearance of 2.4 m and 3.1 m over stagnant and moving water respectfully [32]. Since the early 2000's, de-bonding of corrosion products in layers over driving lanes of box girders has been observed, as well as problematic corrosion at connections of braces over travelled lanes.

The goal of new design and detailing is to prevent the deterioration conditions from occurring. Deterioration of the ACR patina can be expected to occur where there is prolonged time of wetness, which may occur when there is:

- High humidity due to either the general climate or local effects such as surface water or vegetation growing against the bridge.
- Roadway or marine salts which slow the drying process and accelerate corrosion.
- Debris that traps moisture.

Detailing guidance was first introduced in the March 2024 publication of the *Structural Manual* on design and detailing requirements for ACR steel. The principles can generally be applied to rehabilitation and focuses on:

- The general geography in which the bridge is located (macro-climate).
- The local, site-specific characteristics of the bridge (micro-climate).
- Prevention of exposure to high levels of chlorides (Cl<sup>-</sup>).
- Time of wetness.

Repairs for poor patina generally focus on reducing moisture exposure by improving drainage, removing water-retaining debris and vegetation, and cleaning the steel to help it dry more quickly. Where chlorides are present, periodic pressure washing may be needed to reduce contamination. Consult with Structures Office when patina repairs are contemplated.

Additional information on these concepts may be found in the *NSBA Uncoated Weathering Steel Reference Guide* [33].

Significant research has been conducted on the development and performance of the patina of ACR steel, including *NCHRP Report 272 Performance of Weather Steel in Bridges* [34] and *NCHRP Report 314 Guidelines for the use of Weathering Steel in Bridges* [35]. The concepts

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and considerations for atmospheric corrosion of weathering steel are explained in detail in [36] and [37].

### **P2-3.14.3.6 Removing and Replacing Structural Steel**

#### **Locally Removing and Replacing Structural Steel**

The Ministry has replaced segments or entire girders following severe load impacts. The success and durability of this type of 'remove and replace' repairs depend significantly on detailing, including access cut-outs in the web, surface preparation of mating surfaces, transition details between existing and new welds and steels, and quality assurance through non-destructive testing throughout the repairs.

#### **Removing Cross Bracings**

Cross bracing over travelled lanes can be exposed to salt spray and often experiences crevice corrosion. Blasting and coating of poor-performing braces is often difficult. Removal of braces over travelled lanes should be considered, if possible. Removing external intermediate cross bracings can also eliminate out-of-plane distortion for bridges with fatigue prone connection details. An analysis is required to determine the effects of removing cross bracings on the adequacy and serviceability of the steel girders.

### **P2-3.14.3.7 Repairs to Structural Steel Coating**

Guidance pertaining to structural steel coating are documented in the latest version of the Ministry's *Structural Steel Coating Manual*. The Manual outlines considerations for different coating rehabilitation approaches including recoating, overcoating and zone painting.

Overcoating generally describes the practice of painting over an existing coating. The Bridge Office Report BO-99-01 *Steel Bridge Coating Program – Cost and Options Study* [38] evaluated the cost-effectiveness of overcoating versus full removal and recoating. It found that overcoating is generally not cost-effective as a long-term strategy, especially when access and traffic control costs are significant, due to its shorter lifespan compared to recoating. Overcoating might be a suitable approach from a network-level bridge management perspective to allocate limited funding to more bridges. However, for individual rehabilitation projects, zone painting and recoating are preferred due to their more stringent abrasive blast cleaning requirements, which provides better adhesion and performance of the new coating.

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**P2-3.15 Post-Installed Adhesive Anchors into Concrete (Dowels)****P2-3.15.1 Background**

Anchors in concrete are used to transmit structural loads by means of tension, shear, or a combination of tension and shear between connected structural elements or safety-related attachments and structural elements.

There are many types of anchor systems including:

- Cast-in-place:
  - Embedded bolt.
  - L-bolt.
  - J-bolt.
  - Welded headed stud.
  - Shear lugs.
- Post-installed:
  - Bonded:
    - Adhesive.
    - Grouted.
  - Mechanical:
    - Undercut
    - Expansion:
      - Torqued-controlled sleeve.
      - Torqued-controlled wedge.
      - Displacement-controlled wedge.
  - Screw.

Other types not currently covered by codes include anchor channels with test results from ICC-ES AC-232, reinforcing bars welded to plates, coil loops, lead-shield wedge anchors, powder-actuated anchors, and plastic anchors.

Section 12.5.1 in the latest *Structural Manual* (Revision #61, 2025) provides additional information on post-installed adhesive anchors into concrete, including approved products (i.e., DSM List) and limitations in their application.

**P2-3.15.1.1 History**

On July 10, 2006, the suspended ceiling system of the I-90 connector tunnel in Boston collapsed. After the collapse, the Federal Highway Administration (FHWA) conducted an independent investigation and issued *Technical Advisory TA5140.26* on October 17, 2007, strongly discouraging the use of fast set epoxy for adhesive anchor applications or the use of any kind of adhesive anchors for permanent sustained tension applications or overhead applications until an improved certification process had been developed. The advisory *TA5140.30* was updated on March 21, 2008, to update the list of “Fast set epoxies” of *TA5140.26*.

The MTO Engineering Materials Office, Concrete Section maintains a DSM List of adhesives for dowels which previously included both epoxy and acrylic-based products. A limited scope of tests was conducted on the available products in the late 1980’s and the results were documented in

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*Report MI-120* dated April 1988. Between then and May 2008, additional products were accepted for the DSM based on test information submitted by the manufacturer's verifying compliance with the acceptance criteria. The acceptance criteria did not include assessment of creep characteristics of the adhesives under sustained load or performance of embedded dowels in freeze thaw conditions.

In May 2008, the Bridge Office issued a *Bridge Office Design Bulletin – Guidelines for the Use of Dowels Embedded in Epoxy Adhesives* explaining the details above and including recommendations to limit the use of adhesive anchors in general, prohibit the use of fast set epoxies, and limit sustained tension load applications.

Subsequently, significant research and advancements in epoxy technology and anchor installation requirements has been completed, explained in FHWA publication *TA5140.34*, *NCHRP Report 639*, *NCHRP Report 757*, and the ACI/CRSI "Adhesive Anchor Installer" certification program. This work has been incorporated into codes and is now available in *CSA S6-25 CHBDC*, Annex A8.2 as design requirements.

The Ministry's DSM list has been updated but still has a limited testing program and no adhesives have been approved for sustained tension loads or overhead or upwardly inclined installation applications.

While code provisions may allow generic anchor designs that accommodate a wide range of acceptable products, achieving this generality requires the application of large reduction factors, leading to conservative and less efficient designs. This approach, however, aligns with MTO's construction contract framework and bidding practices. Designers should expect contractors to propose alternative adhesive products or anchor systems, and these alternatives will require review during construction. Because adhesive anchors are qualified and tested as complete systems, the Evaluation Service Report and the Manufacturer's Printed Installation Instructions (MPII) must be consulted for the design and installation requirements applicable to the specific system and site conditions. System-specific design may also require the use of proprietary software.

### **P2-3.15.2 Construction Details**

Dowel reinforcement must be installed straight and parallel into the concrete substrate; bending dowels to increase embedment depth alters their intended load transfer and failure mechanisms and is not permitted.

Dowels are not permitted to be installed in concrete less than 21 days old. This requirement is in the construction specification and is a moisture-sensitivity consideration, not a strength requirement and shall be accounted for in working day estimates.

### **P2-3.15.3 Design Requirements**

Anchors post-installed into concrete shall be adhesive bonded dowels. Mechanical, grout bonded, and screw types shall not be permitted for permanent applications, nor other types not covered by any codes.

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Where design codes, including *CSA S6 CHBDC*, provide comprehensive anchor design models, they also require corresponding qualification tests to confirm that the anchor system meets the assumptions of those design provisions. The current MTO DSM does not include all the testing necessary to support the use of these comprehensive design models. Therefore, any post-installed adhesive anchor used in applications other than “down-hole” installation must be a product that has been tested and qualified in accordance with the latest published assessment criteria of *ACI 355.4* or *ICC-ES AC308* for the specific application conditions. Most factors affecting the bond strength of adhesive anchors are controlled during installation. Because proper installation and inspection are critical to their reliable performance, and certified installers and inspectors are not yet mandated by specification, adhesive anchors installed horizontally or upwardly inclined to resist sustained tension loads are not permitted at this time.

The presence of supplementary reinforcement allows the use of higher resistance factors. Specific detailing must be provided for both tension and shear anchor reinforcement, and this reinforcement must be oriented in the direction of the applied load and placed close to the anticipated crack initiation zone. Reinforcement placed perpendicular to the load direction does not act effectively as shear-friction reinforcement and should not be relied upon for this purpose.

Dowels into concrete shall not be larger than 25M.

Short-term bond strength shall not be greater than 50% of the reported value for the selected adhesive.

FRP dowels are not permitted.

**P2-3.15.3.1 Using Dowels to Reconstruct Barrier Walls**

When reconstructing existing barrier walls during rehabilitation, it has become increasingly common to saw-cut the existing barrier wall and reinforcing steel, then dowel in new reinforcing steel. This is faster than retaining and chipping around existing reinforcing bars. The anchor designs shall adhere to *CSA S6-25 CHBDC* requirements.

**P2-3.15.4 OPS Specifications and References**

Installation and acceptance testing shall be according to SSP 999F29 for metallic dowels.

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### P2-3.16 Liners for Structural Culverts

#### P2-3.16.1 Background

Colloquially, culverts spanning greater than 3.0 m are often referred to as “structural culverts” and those spanning less than 3.0 m are referred to as “non-structural” or roadway culverts. Design of buried structures with spans of 3.0 m or more are governed by the Buried Structures section of *CSA S6 CHBDC*. However, buried structures with spans less than 3.0 m are not covered in the code and do not need to adhere to its requirements. *CSA S6 CHBDC* only includes design provisions for concrete, steel and aluminum culvert materials.

#### P2-3.16.2 Construction Details

Culvert lining involves installing a secondary layer (lining) inside an existing culvert to restore or improve its performance. A culvert liner may enhance durability, or it may partially or fully supplement the structural function of the original host culvert. A range of liner types and installation methods exist; however, only certain liners are permitted for structural culverts on Ministry projects. Several products used for smaller diameter non-structural pipes are excluded.

The selection of liner type and material depends on site conditions, hydraulic performance requirements, environmental constraints, and constructability considerations. In many cases, site access and staging limitations strongly influence the installation method. Most liners for structural culverts are installed using slip-lining, where a pre-assembled, smaller-diameter pipe is inserted into the host culvert and the annular space is grouted to transfer loads to the new pipe. For steel insert pipes, additional options such as multi-plate liners or tunnel plates may also be used.

The Ministry has also rehabilitated culverts using both chemically cured and cementitious spray-applied liners. In this method, materials are mixed on-site and sprayed onto the interior surface of the existing culvert to the required thickness, forming a new pipe barrier once cured.

#### P2-3.16.3 Design Requirements

New provisions in *CSA S6-25 CHBDC* require designers to assign exposure classes based on environmental parameters. The code specifies limits for pH, chloride content, resistivity, water-soluble sulphates, and hardness for water around submerged structure, groundwater, and soil. These factors, along with the design service life of the culvert, influence the choice of liner material. In some cases, additional protective measures, such as inert coatings, may be necessary. Chapter 7 (Durability Design Evaluation) of the Ministry’s *Gravity Pipe Design Guideline* provides a methodology to evaluate the design service life of specific materials when the above factors are known.

The liner design shall be completed by the design engineer during the design phase and detailed in the contract documents. Where available, alternative products shall be permitted consistent with the design intent. Liners shall be designed as a standalone structure, without relying on contribution from the original host pipe. The condition of the existing liner shall be considered, and interaction with the host pipe shall be accounted for in the boundary conditions.

For liners that fall outside the scope of *CSA S6 CHBDC*, alternative design criteria require approval.

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**P2-3.16.3.1 Corrugated Steel Pipe (CSP) and Structural Plate Corrugated Steel Pipe (SPCSP) Liners**

Standard CSP culverts are available in both round and pipe-arch shapes. They are lightweight and readily available. Round CSP can be supplied up to 3.6 m in diameter, and 4.2 mm in nominal thickness, while larger sizes are available for pipe-arch configurations. However, the corrugations reduce hydraulic efficiency, especially when CSP is used as a liner where the internal diameter is already constrained.

CSP are protected against corrosion and abrasion using a galvanized coating. Additional protective measures are available, including polymer-laminated coating. For these, flat steel sheets are first galvanized and then laminated with a polymer layer on both sides before being formed into corrugated steel pipe.

SPCSP consists of heavy-gauge corrugated steel plates that are bolted together on-site to form large-diameter culverts, arches, or custom shapes. The increased plate thickness and deep corrugations provide additional structural capacity in applications where conventional CSP is not feasible.

SPCSP can be galvanized or for additional protection, it can be coated with a thermoplastic ethylene acrylic copolymer with a zinc primer. Due to their significantly thicker steel plates, SPCSP culverts can accept a heavier zinc coating compared to galvanized CSP culverts, providing greater sacrificial protection than similarly coated thin-gauge CSP. The thicker steel plates used in SPCSP can be properly deburred and prepared before coating, which promotes stronger adhesion and reliable protection at the plate ends. In contrast, the ends of coated CSP often represent a weak point in the coating system and may be more susceptible to peeling.

CSA *G401* covers the standards for corrugated steel pipe and buried structures for culvert and a range of related drainage and hydraulic applications.

**P2-3.16.3.2 Welded Smooth-Walled Plates**

Thick carbon steel plates are available in the North American market and can be custom bent to suit existing culvert geometry. The plate segments are inserted using slip-lining method and then welded together in place. This approach is structurally robust and can address service life requirements through sacrificial steel thickness.

**P2-3.16.3.3 Aluminum Culvert Liners**

Aluminum culverts are lightweight and manufactured from corrugated aluminum alloy sheets. Their natural resistance to corrosion makes them well-suited for severe corrosive environments. However, aluminum culverts are generally cost-prohibitive compared to other alternatives and are not commonly used on MTO projects.

**P2-3.16.3.4 High Density Polyethylene (HDPE) Liners**

High-density polyethylene liners are commonly used for culvert rehabilitation using slip-lining. HDPE is lightweight and easy to handle, and it is available with corrugated and smooth interior

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and exterior walls. Smooth interior walls can improve hydraulic performance. HDPE liners are best suited for circular culverts, where their geometry minimizes the amount of required grouting.

Their diameters limit their usefulness for lining larger structural culverts; in some cases, two smaller HDPE pipes were fitted within a single large culvert to compensate. HDPE is often more appropriate for temporary bypass or short-term applications rather than permanent structural relining of large culverts.

HDPE liners shall not be relied on for structural capacity, unless approved by the owner.

### **P2-3.16.3.5 Fibre Reinforced Polymer (FRP) Liners**

FRP liners can be custom-made to match the existing culvert geometry, making them particularly useful for rehabilitating culverts with irregular, deformed or non-standard shapes. The liners are typically supplied in 2.4 m (8 ft) long sections with socket-and-spigot joints. FRP liners have lower stiffness and long-term creep behaviour.

FRP liners shall not be relied on for structural capacity, unless approved by the owner.

### **P2-3.16.3.6 Spray-Applied Liners**

Sprayed-on liners provide minimal reduction to the opening and conforms to the shape of the existing culvert. This makes them more effective in handling distortions or irregularities in the existing culvert geometry, which can pose challenges for inserting traditional pipe liners.

Spray applied liners shall not be relied on for structural capacity, unless approved by the owner.

### **P2-3.16.4 OPS Specifications and References**

OPS construction specifications for pipe culverts, such as *OPSS.PROV 420* and *421*, are intended for roadway culverts and not applicable to structural culverts. Structural liner applications shall be captured by Non-standard special provision.

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**P2-3.17 Deck Top Rehabilitation using Short-Term Closures****P2-3.17.1 Background**

Short-duration nightly closures are increasingly used for rehabilitation projects, especially in urban areas where long-term continuous closures are not feasible. These limited work windows significantly affect deck top rehabilitation activities, including patch repairs, waterproofing, and joint replacement. The challenge is to sequence the work so that the deck can be restored to a safe and serviceable condition at the end of each closure.

These “rapid” rehabilitations often involve methods, materials, and staging approaches that deviate from conventional practice and carry higher uncertainty. Designers must recognize from the outset that accelerated construction techniques, though necessary to meet restrictive closure windows, may not achieve the service life as traditional rehabilitation. Both the immediate operational constraints and the long-term performance implications must be carefully considered. Early coordination with the appropriate Ministry offices (Construction, Operations, Engineering Materials Office, and Structures Office) is essential.

**P2-3.17.2 Construction Details****P2-3.17.2.1 Patch Repairs**

When it is known during design that normal concrete is not feasible and proprietary materials may need to be specified for patch repairs, they shall be included as a separate tender item. These products can offer advantages suited to restrictive working windows, such as trowel application, self-levelling, and rapid-hardening formulations, making them useful for small, time-sensitive repairs. However, their use requires careful evaluation. Proprietary materials are typically supplied in small bagged quantities, have limited working time, and are labour-intensive to place, which makes them challenging for larger or deeper repair areas commonly encountered in deck rehabilitation. Many formulations perform as mortars unless extended with approved aggregates, and thick placements can increase the risk of shrinkage or thermal cracking. Some products also lack adequate abrasion resistance and are not suitable for surfaces exposed to traffic.

**P2-3.17.2.2 Waterproofing**

Newly placed waterproofing cannot be exposed to traffic until it is overlaid with asphalt. The membrane must cure and cool adequately before paving, which further extends the required closure window. Under rapid-repair scenarios, work may need to be staged in smaller, manageable segments to maintain traffic flow, or an alternative waterproofing system may be required. The use of alternative waterproofing systems is evaluated on a case-by-case basis, with approval from Structures Office.

**P2-3.17.2.3 Expansion Joint Replacements**

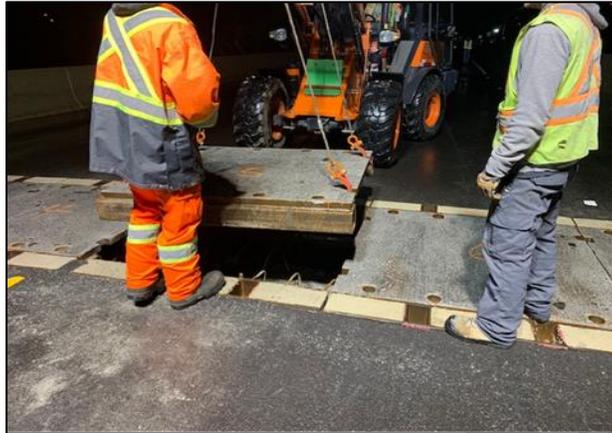
The Ministry has successfully completed a number of expansion joint replacements using “bridging plates”. This scheme was first used around 2007 to replace joints on the QEW Burlington Bay Skyway Niagara-bound lanes. Since then, their use has become more common on Ministry projects.

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Temporary removable steel plates are installed spanning over the existing joint with temporary asphalt ramp on each side, allowing traffic to drive over them safely. When lanes are closed, the panels can be removed quickly, providing the contractor with access to the joint to complete the work, [Picture 37](#).



**Picture 37 – Removing bridging plate to access the expansion joint during a nightly closure.**

The initial installation and final removal of the bridging plates and temporary asphalt ramps are the most time-sensitive and critical operations. Bridging plates must be installed across the full width of the structure to avoid creating an unsafe drop-off. The initial installation and final removal typically require extended closures, often over an entire weekend. Once the plates and temporary asphalt are installed, the joint work is completed over nightly closures. In this scheme it is feasible to use normal concrete in the reconstructed end dams.

On some projects, where extended closures are not available for setup and take down, the temporary asphalt ramps are placed first over several nightly closures, with temporary asphalt directly placed over the existing joints. Once the ramps are in place across the full width, the asphalt over the joints is removed, and the bridging plates are installed, and staged over multiple closures as required.

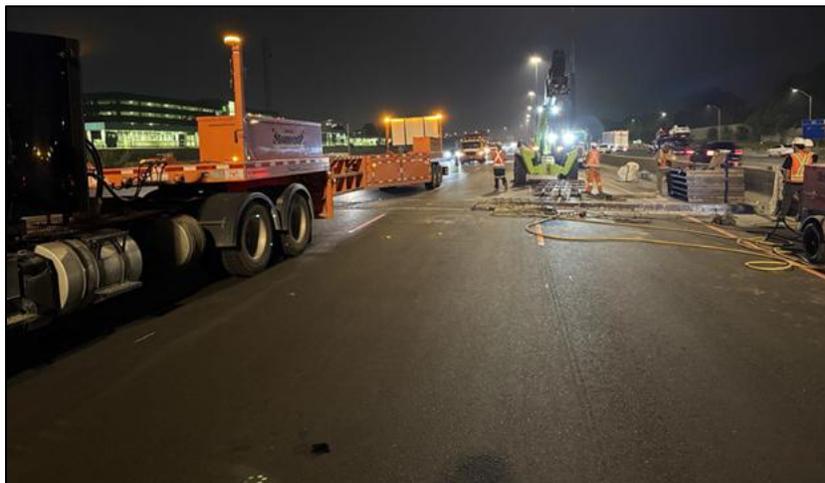
### **P2-3.17.3 Design Requirements**

Rapid repairs completed within short closure windows are not always efficient when viewed over the full rehabilitation cycle. These treatments often require substantial temporary works and additional construction joints, which can extend overall project duration, increase costs, and introduce long-term maintenance implications.

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**Picture 38 – Nightly closure using mobile barrier.**

To maximize the limited working window, designers must consider alternatives to traditional temporary construction barriers, which require significant time to install and remove. Solutions such as quick-change moveable barriers or truck-mounted mobile barriers ([Picture 38](#)) can improve efficiency, but each has constraints that must be weighed early in design. Quick-change “zipper” barriers allow rapid lane reconfiguration but have high mobilization costs. Mobile barrier systems provide fast set up, but their limited availability and fixed lengths may necessitate staging the work in smaller segments. These factors must be evaluated early in the design process to ensure the selected traffic protection strategy aligns with project constraints and operational windows.

Accurately assessing the deck condition during design is critical. Additional investigations beyond the minimum requirements of Section [P1-2 Levels of Condition Assessment](#) may be necessary to refine repair quantities and mitigate the risk of significant overruns during construction.

These projects often require an hour-by-hour breakdown of the most critical operations to ensure work can be completed within restrictive closure periods. Early discussions with Traffic, Construction, Operations and relevant offices are essential to confirm the feasibility of proposed sequencing, lane closure durations, and traffic management strategies. Similarly, designers must consult with the Engineering Materials Office, Concrete Section to confirm that selected materials, particularly rapid-setting or accelerated-cure products that fall outside of the scope of OPS specifications, are suitable for the exposure conditions, performance requirements, and allowable construction windows.

These projects generally carry greater uncertainty in their long-term performance, and therefore must be considered holistically. Rapid repair solutions may not offer the same durability or service life as more conventional rehabilitation, and designers must carefully assess whether a short-term, accelerated approach is appropriate, or whether a more disruptive but comprehensive intervention is warranted to avoid repeated follow-up work.

#### **P2-3.17.4 OPS Specifications and References**

By nature, these projects deviate from the requirements outlined in the standard OPS Specifications. The OPS Specifications listed in [Table 34](#) are provided for reference and apply

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only to the general work associated with these operations. The 'rapid' deck top repair methods described in this section are not directly covered by those Specifications.

Early consultation with the relevant offices is essential to confirm acceptable approaches, identify project-specific requirements, and ensure that any departures from OPS requirements are properly justified and documented.

**Table 34 – OPS Specifications and References for 'Rapid' Deck Top Repairs.**

<b>Means and Methods</b>	<b>OPS Specifications</b>	<b>NSSP (Custodian Offices)</b>	<b>CDED Chapter</b>
Patch Repairs under Short-Term Closures	OPSS.PROV.930	Project-specific NSSP required. (Consult EMO, Concrete Section and Structures Office)	-
Using Alternative Waterproofing System	OPSS.PROV 914	Project-specific NSSP required (Consult EMO, Concrete Section and Structures Office)	-
Expansion Joint Replacement using Bridging Plates	OPSS.PROV 920	Project-specific NSSP required. (Consult Structures Office)	-

A comprehensive repository of MTO technical documents can be found on the [MTO Technical Publications](#) website.

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