Government of Northwest Territories
Gouvernement des Territoires du Nord-Ouest

STRUCTURES (BRIDGES) DESIGN GUIDELINES

Version 1.0

December 2024

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Government of Northwest Territories

Structures (Bridges) Design Guidelines

Version 1.0

2024

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STRUCTURES (BRIDGES)

DESIGN GUIDELINES

Department of Infrastructure

Government of Northwest Territories

Version 1.0

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PREFACE

The Government of Northwest Territories (GNWT) Structures (Bridges) Design Guidelines (SBDG) documents the design practices and policies followed by the GNWT Department of Infrastructure (Department) with respect to highway bridges and related structures and is intended to supplement and supersede the requirements of the Canadian Highway Bridge Design Code CSA S6:19 (CHBDC).

While the document is primarily focused on structural engineering issues, it also touches upon conceptual design issues, drafting standards and material specifications. As a result, certain components in the document overlap with other Department documents. The Department strives to provide consistency between these documents, however, occasionally changes to one document may not be immediately reflected in other documents. If a discrepancy is found, the Consultant should request clarification from the Department.

Standard requirements pertaining to geometry, detailing, and materials are included with the goal of producing reasonably uniform design products from a range of Consultants. In the experience of the Department, these requirements have been found to reduce design, construction, inspection, and maintenance issues, while providing an acceptable balance between safety, quality, and cost.

It is not the intent of the document to limit progress or discourage innovation; Consultants are encouraged to explore all engineering options they deem appropriate for a specific site. Designs that are not in accordance with this document shall not progress past the preliminary engineering stage unless Approved.

LIST OF CHANGES

The table below documents the timeline of changes to the Structure Design Criteria.

When changes are made to the document, the following actions will be completed:

- The version number of the document will be updated.
- A revision triangle will be placed next to the change in the document (for minor revisions following the initial issue of the document).
- A basic description and the date of the change will be summarized in the table below.

VERSION NUMBER	DATE	DESCRIPTION

Table of Contents

List of Tables

1. INTRODUCTION

1.1 General

These Structures (Bridges) Design Guidelines (SBDG) apply to the design of highway and pedestrian bridge structures, bridge-sized culverts, overhead sign structures, and earth retaining structures.

These guidelines have been written in the context of bridges over watercourses. While the information is equally applicable to bridges over roadways and railways, those types of structures have additional features and requirements that are not covered by these guidelines. For bridges over roadways and railways, the Consultant shall supplement these guidelines with relevant information from the latest version of the Alberta Transportation Department of Transportation and Economic Corridors (TEC) Bridge Structures Design Criteria, the British Columbia Ministry of Transportation and Transit (MOTT) Bridge Standards and Procedures Manual Vol. 1, Supplement to CHBDC S6:19 (MOTI, 2022), and documents as detailed in these SBDG and documents from other jurisdictions as agreed upon with the Department.

Unless noted otherwise in the SBDG, all structures shall be designed in accordance with "CSA S6:19 Canadian Highway Bridge Design Code" (CHBDC). The provisions in the SBDG shall govern over the provisions in the CHBDC.

Exceptions to the design requirements set out in the SBDG may be justified under exceptional circumstances. Any such design exceptions shall be fully documented by the Consultant and submitted in writing for review and Approval by the Department. This process also applies to requirements for Approval by the owner or regulatory authority that are included in the CHBDC.

Wherever these guidelines refer to other documents and publications, the latest version of those documents shall be used unless specifically stated otherwise.

1.2 Bridge Design in Northern Climates

Bridge and culvert design in northern Canada and cold climates need to consider conditions that may not be normally encountered in southern parts of the country, including extremely low temperatures. The following sections briefly summarize some general constraints and challenges that designers should be aware of and consider during project planning, design, tendering, and construction.

1.2.1 Construction Season

The north has shorter summer and longer winter seasons. If access is only available during winter, bridge design should consider winter construction methods and techniques (see Section 1.2.6 (Winter Construction)).

1.2.2 Remote Location

Many project sites are located in remote and isolated areas or along winter roads that are only accessible during the winter season. This leads to logistical challenges and constraints for construction as-well as other field work such as data collection, environmental assessments, geotechnical investigations, topographic surveys, and other site work. Fly-in or barge access may be the only options. The remoteness of the site should be considered throughout the design phases to assure the structure is robust, low maintenance and cost effective. Remote sites with limited or seasonal access, will have an impact on construction costs due to high mobilization/demobilization and transportation requirements. The GNWT regional offices may not be nearby, therefore, support from the Department may be delayed or not available in a timely manner.

1.2.3 Low Maintenance Structures

Bridges and culverts should aim to be robust and low maintenance due to the limited resources available in the north to complete repairs and even routine maintenance activities. Considerations to improve durability and reduce the required maintenance on structures include but are not limited to the following:

- Incorporating good drainage practices in the bridge design is critical to prevent water and ice accumulation and related issues. Water must be directed away from the structure and critical components wherever possible. Proper drainage design will mitigate corrosion, embankment instability, and erosion, while also mitigating icing conditions and hydroplaning due to ponding water.
- De-icing salts may be used on northern roads, therefore protection from corrosion and the use of corrosion resistant materials is important for long-term durability.
- Effective riprap design and placement practices is crucial to mitigate scour and erosion around bridge foundations, streambanks, and culvert inlets / outlets. Erosion and scour often leads to one of the most common and costly long-term maintenance needs at crossings.
- Timber decks are prone to rapid wear and tear due to winter chains often used by commercial vehicles. Steel grate decks are not desirable but may be Approved for replacement of existing timber decks where the increase in weight associated with a concrete deck cannot be accommodated.

1.2.4 Thermal Movement Considerations

Some Northwest Territories regions and bridge sites experience sun rising low on the horizon where only one side of the bridge/girders is exposed to long hours of sunshine, resulting in differential thermal expansion between the side of the structure exposed to the sun and the other side which remains relatively cold. The Department has observed that due to excessive and irregular thermal expansion, girders have walked off bearing pads, punched into backwalls, and damaged longitudinal restraints. In addition to the differential noted between parts of the structure, the effective temperature of the structure as compared to the mean daily air temperature may differ more than Table 3.8 and Figure 3.5 of the CHBDC would suggest.

The Consultant’s attention is drawn to account for additional thermal expansion based on the region and orientation of the structure. Structures in the Mackenzie Valley appear to experience more of this thermal effect (also called “solarity”) as compared to the South Slave Region.

The differential girder expansion also appears to result in rotational movements in bridge bearings on the side exposed to the sun, which impacts the warmer side bearing more than the colder side. Bridge articulation should be carefully considered to accommodate these effects.

Due to lack of research data available for both aforementioned impacts, no specific allowance is assigned to account for these impacts. Designers are encouraged to engage the Department for additional guidance and experience related to specific regions and structure orientations.

1.2.5 Design Considerations

The Consultant should not assume that contractors in the north have the same range of equipment, level of experience, and proficiency that might be expected in southern Canada. Structures should be kept as simple and straightforward as possible to maximize the number of contractors who may be able to complete the work.

The use of prefabricated elements should be considered whenever possible, allowing the work to be carried out in a controlled shop environment and to minimize the time required on site.

Cast-in-place concrete relies on the presence of batching facilities within a reasonable distance from the project site. Designs that use materials such as precast concrete or structural steel are preferred as they are not reliant on these facilities.

Transportation limitations must also be considered by the Consultant, particularly when winter roads are involved. These limitations may include restrictions on the length, width, height, and weight of payloads that can be moved to site. The ability to transport girders that are longer than a standard highway vehicle must be confirmed, and these girders may have to be carried horizontally to ensure the stability of the truck. It is important that any such transportation restrictions are identified by the Consultant early in the design process.

1.2.6 Winter Construction

The short summer and logistical constraints in the Northwest Territories result in winter construction occurring to a greater degree and under more severe conditions than would be typical in southern Canada. However, there are limitations to what can realistically be achieved, in particular with earthworks during the winter. This impacts approach fills, buried structures and earth-retaining structures.

The primary constraint is that it is not feasible to moisture-condition fill materials during placement and compaction in the winter. Therefore, specified levels of compaction may not be achievable and settlement of fill placed in the winter should be expected.

Several methods can be used to limit the potential magnitude of settlement:

- Clean, coarse aggregate is not moisture-sensitive so could be considered if available.
- Any fill material placed in the winter should have a moisture content less than optimum.
- Fill containing visible ice should not be used in applications where settlement can not be tolerated.
- Lift thicknesses should be reduced from what would be acceptable during summer construction for a given piece of equipment.
- Overbuild could be considered in some applications, either in addition or as an alternative to the foregoing. This would require allowance for thaw and settlement to occur during the following years.

From a structural perspective, it is important that the expected construction schedule is considered during the design stage. This allows the designer to make appropriate decisions in terms of material choice and other design issues - determining the likely setting temperature for bearings and joints, for example.

1.2.7 Permafrost

If any aspect of the design is to interact with permafrost, a permafrost specialist should be engaged to provide advice in those areas. It is normally preferable to avoid permafrost where practical, particularly ice-rich permafrost. For example, even at a site underlain by permafrost, if the presence of sound (non-ice-rich) bedrock at reasonable depth can be confirmed, then it would be sensible to extend the foundation to bedrock. However, approach fills would still be constructed over permafrost, so a permafrost specialist should be consulted to consider stability and deformation aspects of the fill construction.

A geotechnical site investigation must be completed by the Consultant in advance of design, unless provided by the Department. Additional parameters that are required from an investigation of permafrost, compared to a conventional geotechnical investigation, are: ground temperature, ice content, and soil porewater salinity. If permafrost is present at a site, the geotechnical data should be reviewed to determine whether the data collected is sufficient for the project or if further investigation is required. CAN/BNQ 2501-500/2017 (Geotechnical Site Investigation for Building Foundations in Permafrost Zones) is a standard that sets out requirements for geotechnical investigations of permafrost sites.

Of the foregoing parameters identified for characterizing permafrost, ground temperature is arguably the most important. Ground temperatures collected from drill cuttings are not considered to be satisfactory. Ground temperature is not a single value for a particular site but varies spatially and temporally. Therefore, the installation of instrumentation that permits ground temperatures to be measured over a range of depths and times is required. Data collection must be sufficient to permit analyses to be conducted that will allow ground temperatures over the life of the structure to be predicted.

If the site conditions are such that founding the structure on permafrost soil cannot be avoided, it must be recognized that the design approach requires quantification of anticipated settlements over the life of the structure. The design of foundation capacity on ice-rich permafrost is governed by creep theory, such that the magnitude and duration of the loading, as well as the magnitude of permissible deformation, must be considered. As such, foundation capacity is not normally governed by ultimate limit states but serviceability limit states conditions. The Canadian Foundation Engineering Manual (CGS 2023) provides basic information on design considerations for foundations on permafrost.

It is generally accepted that the design of any infrastructure on permafrost should consider the impact of climate warming on the characteristics and integrity of the permafrost over the life of the structure. The CHBDC outlines a risk-based approach to assessing the rigour of analysis that is warranted. This risk-based methodology complements the consequence-level definitions of the CHBDC.

Even with good baseline data and rigorous analysis, it is difficult to reliably predict permafrost integrity and associated permafrost performance over the 75-year design life mandated by the CHBDC. Therefore, consideration should be given to adopting some form of adaptive design that predicts potential service limitations and provides a means to address them. The adaptive design approach should be documented and Approved. Additionally, performance monitoring of key parameters (e.g., deformation, ground temperature) over the life of the structure is imperative.

1.2.8 Aufeis

Culverts and bridge openings in the north may be prone to icing and the development of aufeis (Ensom et. al., 2020). This build-up of ice can result in the blockage of culverts and bridge openings, creating drainage issues during spring melt. For example, the build-up of ice throughout the winter can result in initial spring runoff and associated scour occurring at a higher elevation under a bridge than would be predicted by conventional open-water hydraulics.

Overflow culverts outside of the natural channel placed at higher elevation or lateral distance away should be considered when aufeis/icing concerns are noted at existing or adjacent crossing sites. The overflow culvert will not mitigate the growth of aufeis but may provide a secondary drainage path for spring runoff, while the deeper culvert or bridge opening is still blocked. Other icing mitigation such as steam pipes should also be considered as part of the design stage to allow highway maintenance crews to thaw frozen culverts ahead of spring runoff.

1.2.9 Northwest Territories Manufactured Products Policy

The objectives of the Northwest Territories Manufactured Products Policy are to:

- Encourage local production as a means to diversify the NWT economy; and
- Foster and maintain the investment, jobs and income produced by local manufacturing.

Additional information on the policy can be found at the following link:

https://www.iti.gov.nt.ca/en/services/nwt-manufactured-products-policy

Wherever possible, the Consultant should consider the applicability of this policy throughout the development of the design drawings and technical specifications.

1.3 Regulatory, Archaeological, and Environmental Considerations

1.3.1 Regulatory Considerations

Early evaluation of the necessary regulatory approvals determines the scope of biophysical, socio-economic, and cultural components that require assessment (Section 1.3.4 (Environmental Considerations)). Appropriate seasons for investigations and timelines need to be considered for approval reviews to facilitate appropriate scheduling for the design process.

The Consultant must consider all applicable federal and territorial legislation and regulations concerning environmental protection, including but not necessarily limited to the following:

- Northwest Territories Lands Act
- Mackenzie Valley Resource Management Act
- Waters Act
- NWT Species at Risk Act
- Environmental Protection Act
- Fisheries Act
- Canadian Navigable Waters Act
- Migratory Birds Convention Act

- Species at Risk Act

The Consultant must consider the scope of the project and determine all approvals and authorizations that may be necessary and schedule the application process to allow adequate time for engagement requirements and agency review.

Federal, territorial, and municipal legislation can and will change over time. Therefore, when reviewing data sources and background information, the date and source of the data should be considered to ensure that the information used to support conceptual design is applicable and aligns with current regulatory requirements. Early contact with regulatory agencies is encouraged to confirm the scope and schedule for each application.

1.3.2 Indigenous Considerations and Engagement

The Mackenzie Valley Resource Management Act requires that the concerns of Aboriginal people and the general public are taken into account through the regulatory process (paragraph 114(c)). The Act grants authority to the Land and Water Boards of the Mackenzie Valley to regulate the use of land and water. The Boards must ensure that the concerns of Aboriginal people and the public are considered, and that their decisions have regard for the protection of the social, cultural, and economic well-being of residents of the Mackenzie Valley. The Engagement and Consultation Policy reflects standards for meaningful consultation (as set out by the land claims and applicable legislation) with all affected parties, including Aboriginal groups in the Mackenzie Valley, are met and clearly articulated.

Engagement with Indigenous governments, organizations and others is encouraged early in the planning process to consider conservation for the well-being and way of life of Aboriginal peoples of Canada. Proponents should work to ensure potential impacts of proposed projects are understood and carefully considered before design decisions are made. Proponents must work to understand the full nature of concerns expressed by affected parties and improve design or consider potential mitigation of these impacts by jointly working to resolve issues.

Requirements for regulatory applications include an engagement plan and record of engagement activities that reflect opportunities to meaningfully contribute to the assessment of impacts on the environment and the establishment of appropriate mitigation. Assessments may also reflect components of the human environment, including cultural significance; historic, current and future land uses; heritage resources; sensitive sites or features in the project area (such as archaeological, historical, or burial sites, spiritual places, trails, special landscape features); and other traditional knowledge. Proponents are required to meaningfully incorporate traditional knowledge into assessments provided and to follow local protocols to collect, use, interpret, and protect this information.

1.3.3 Archaeological Considerations

Archaeological sites in the Northwest Territories are protected by law under the Northwest Territories Archaeological Sites Regulations and are subject to land use reviews. In partnership with land claim authorities and regulatory agencies, the Prince of Wales Northern Heritage Centre (PWNHC) is the agency that oversees the protection and management of archaeological sites in the Northwest Territories.

Guidelines are in place to ensure that archaeological sites are assessed and mitigated before ground disturbance occurs. As part of the design process, the Consultant must retain a qualified archaeologist to conduct an assessment and provide recommendations to avoid or mitigate potential impacts on the archaeological sites or artifacts prior to altering the ground surface. An archaeological overview study must be conducted during the design process to determine the potential for archaeological sites and/or artifacts and to inform the scope of reconnaissance studies and Archaeological Impact Assessments if they are required. This includes a review of known sites through the Northwest Territories Archaeological Sites Database maintained by PWNHC. Prior to any survey for archaeological sites or artifacts, a permit (Class 1 or Class 2) must be obtained by a qualified archaeologist with enough time to accommodate the referral and approval process. Once a permit is in hand, field surveys must be conducted during snow-free conditions and meet all permit conditions.

Refer to the PWNHC publication "Guidelines for Developers for the Protection of Archaeological Sites in the Northwest Territories" for requirements of this process. This publication can be found at the following link, or by contacting PWNHC directly:

> https://reviewboard.ca/upload/project_document/EA1415-02_Guidelines_for_Developers_for_the_Protection_of_Archaeological_Sites_in_teh_NWT.PDF

1.3.4 Environmental Considerations

Identification and consideration of known and potential environmental sensitivities early in the preliminary engineering or conceptual design stages is important to identify suitable crossing locations, avoid unnecessary environmental impacts, and can support selection of the most appropriate structural concept to proceed with through design stages. Early identification of environmental sensitivities helps to confirm the required regulatory permit and approval applications, identify anticipated construction impacts and mitigation measures, and confirm any location or species-specific timing restrictions or constraints that may have implications on design, construction, and the overall project schedule.

Consideration of environmental sensitivities over the anticipated lifespan of the structure, including climate change and the potential for changing local environmental conditions, must occur when planning and designing the structure. Detailed options analysis during the preliminary engineering stage leads to greater clarity on the detailed design, anticipated project schedules and preparation of construction specifications.

The typical steps to determine environmental sensitivities and design implications include:

- Data collection and background information review.
- Site assessments and targeted surveys.
- Identify project component interactions with environmental sensitivities.
- Develop mitigation measures or design modifications to reduce potential impacts.
- Identify residual impacts.

1.3.4.1 Data Collection and Background Information Review

Existing and historical information on known and potential environmental sensitivities, species, and habitats present must be obtained specifically for the project site, as well as for the adjacent and downstream environments. The Consultant is responsible for obtaining and reviewing sufficient data for each project. Some sources that may be consulted for data and background information include, but may not be limited to the following:

- Online government databases
- Aerial imagery

- Environmental reports and assessments
- Fisheries and wildlife studies
- Geotechnical reports
- Hydrotechnical reports
- Roadway reports
- Functional planning studies
- Topographic surveys
- Railway crossing agreements
- Bridge or Crossing Assessments, design and construction reports or drawings (existing crossings).

1.3.4.2 Site Assessments and Targeted Surveys

After background data has been compiled and reviewed, site-specific assessments and/or targeted surveys must be completed to address any data gaps identified during the data collection and background review stages. These surveys are intended to verify historical and existing data and determine any additional site-specific environmental sensitivities present or with the potential to be present. Aquatic, wildlife and terrestrial assessments and surveys may be required to document existing conditions and habitat potential at and adjacent to the structure location. Information gathered during the assessments will be used to support regulatory permit and approval requirements and must be detailed enough to help assess the feasibility and constructability of design options.

Aquatic assessments will need to assess meso- and microhabitat, instream habitat features, channel bed and bank substrate composition, flow depths, patterns and velocities and assess the potential for rearing, foraging, spawning, and overwintering habitat. Special attention should be paid to side or back channels, snyes, deeper pools and instream habitat features such as woody debris and aquatic habitat to help in the evaluation of the potential impacts on the aquatic environment and species present. Assessments should note areas of scour, erosion, and bank instability and assess the adjacent banks for signs of flooding to support design. Fish and/or other aquatic species inventories consisting of multiple capture methods may be required to identify species and life stages present. Determination of fish passage parameters is a critical component requiring biological, hydraulic, and engineering expertise to confirm and meet regulatory purposes. Baseline water quality parameters are also a consideration at this stage to understand variability of natural conditions and will help develop effective mitigation measures to maintain water quality.

Terrestrial assessments need to include adjacent riparian habitat, vegetation communities, canopy cover and topography adjacent to the watercourse. Assessments should identify habitat potential and include an assessment of the anticipated project footprint and adjacent areas for active and abandoned wildlife habitat features (e.g., dens and nests). Active dens and nests are protected under legislation and wildlife species with territorial and federal protection status may have species-specific setback distances and seasonal timing restrictions. Terrestrial assessments targeting specific species may have limitations on when they can be completed. The Consultant is responsible for identifying specific protocols and timing windows for the completion of terrestrial surveys.

1.3.4.3 Identify Project Component Interactions with Environmental Sensitivities

Following the collection of data and site assessments, subject matter experts must review conceptual designs and the anticipated construction activities required to complete each concept to interpret potential interactions of project components with known and potential environmental sensitivities identified. The effect of project interactions with environmental sensitivities must be evaluated and the direction, extent, magnitude, duration, confidence, reversibility, and frequency of occurrence for each of the sensitivities identified. Subject matter experts must determine whether project interactions directly or indirectly affect the environment and must consider the temporary and permanent effects for each sensitivity identified. Data deficiencies, information gaps or limitations, and assumptions must be described when evaluating the potential project effects as the effects on each environmental sensitivity will be the primary focus for regulatory reviewers when obtaining project permits and approvals.

1.3.4.4 Develop Mitigation Measures or Design Modifications

The Consultant must develop general mitigation measures and determine industry standard mitigation measures necessary to avoid and reduce potential effects. These may be recommended for design (e.g., to facilitate fish and wildlife passage) or associated with each activity and stage of construction. Location and required species-specific mitigation measures will depend on environmental conditions, species present and their life stages at and adjacent to the crossing location.

It is important that the constructability and feasibility of incorporating and implementing mitigation measures into the project activities are reviewed and assessed by the design team prior to the preparation of the regulatory permit and approval applications. Mitigation measures proposed in regulatory permit and approval conditions will become part of the conditions and requirements of the project.

Any potential project impacts identified that cannot be sufficiently mitigated are considered residual impacts. Residual impacts may require additional permits and approvals or may require offsetting or compensation.

1.4 Climate Change

The effects of climate change shall be considered in the design of all new structures.

Estimation of climate change impacts should consider data and results derived from the Coupled Model Intercomparison Project Phase 5 (CMIP5) RCP4.5, RCP6.0 and RCP8.5 scenarios, or alternatively, the more recent Coupled Model Intercomparison Project Phase 6 (CMIP6) SSP2-4.5, SSP4-6.0 and SSP5-8.5 scenarios as model output becomes more readily available. Note that the CHBDC will integrate climate change more explicitly in the 2025 edition of the CHBDC, which will be based on RCP 6.0 as the minimum basis for design provisions impacted by climate change.

Climate change effects on environmental loads (e.g., wind and thermal) shall be assessed and included in design calculations where relevant. Some parameters require specialist input as the potential impacts are not directly extractable from climate change models.

Refer to Section 4.1.2.2 (Climate Change Assessment) for guidance concerning hydrotechnical analysis.

1.5 Department Documents

The Department has a number of other design documents that are relevant and shall be used in conjunction with this document, including but not limited to:

- Standard Specifications for Bridge Construction (SSBC).
- Engineering Drafting Guidelines for Highway and Bridge Projects (Drafting Guidelines).

The following Department documents can be found on the Department's website. The latest version of these documents shall always be used.

Standard Specifications for Bridge Construction (SSBC):

https://www.inf.gov.nt.ca/sites/inf/files/resources/gnwt_ssbc_ed_1_2021_final_1.pdf

Standard Specifications for Highway Construction (SSHC):

https://www.inf.gov.nt.ca/sites/inf/files/resources/combined_standard_specification_for_highway_construction_20210331.pdf

Engineering Drafting Guidelines for Highway and Bridge Projects (Drafting Guidelines):

https://www.inf.gov.nt.ca/en/bridges-design-and-construction

Erosion and Sediment Control Manual

https://www.inf.gov.nt.ca/sites/inf/files/resources/dot_erosion_and_sediment_control_manual_-_mar_31_16.pdf

1.6 Other Agency Documents

A number of Alberta Transportation and Economic Corridors (TEC) (formerly Alberta Transportation) and British Columbia Ministry of Transportation and Transit (MOTT) manuals, bulletins and documents are referenced throughout this document and can be found at the following links:

Alberta TEC:

https://www.alberta.ca/bridges-structures.aspx

BC MOTT:

https://www2.gov.bc.ca/gov/content/governments/organizational-structure/ministries-organizations/ministries/transportation-and-infrastructure

1.7 Products

Where generic product types are referred to in this document, the Consultant shall first refer to the SSBC to determine whether one or more specific products are identified in that document.

If the SSBC do not identify an appropriate specific product, the Consultant shall refer to the TEC Products List and/or the MOTT Recognized Products List in order to identify an acceptable product.

TEC Product List:

https://www.alberta.ca/bridges-and-structures-materials

BC MOTT Recognized Products List:

https://www2.gov.bc.ca/gov/content/transportation/transportation-infrastructure/engineering-standards-guidelines/recognized-products-list

If neither of these lists includes an appropriate product, the Consultant shall obtain Approval for the product to be specified.

1.8 Attachments

Unless otherwise Approved, no objects shall be attached to any part of a highway bridge, pedestrian bridge, culvert, or earth retaining structure (e.g., utilities, lighting, signs).

1.9 Department Drawings

The Department has a number of Standard Drawings and Typical Detail Drawings which are referred to within this document. As these drawings are occasionally updated, Consultants must ensure they are using the latest version of all drawings by obtaining them directly from the Department website at the link below:

https://www.inf.gov.nt.ca/en/bridges-design-and-construction

1.9.1 Standard Drawings

Standard Drawings have a prefix "S" in front of the drawing number, these are engineered documents. The Consultant shall refer directly to these documents on the project Detailed Design Drawings and shall include them in the tender drawing set. These Standard Drawings often require additional project specific engineering and detailing which the Consultant must include on the Detailed Design Drawings. Standard Drawings are occasionally updated, and Consultants shall ensure they are including the latest version of the drawings in their tender set.

1.9.2 Typical Detail Drawings

Typical Detail Drawings have a prefix "T" in front of the drawing number. These are not engineered documents and should not be directly included in the tender drawing set. These drawings are provided to illustrate details that are preferred by the Department. Consultants must utilize the preferred details unless otherwise Approved. Consultants are fully responsible to professionally design and draft all details on the project Detailed Design Drawings.

1.9.3 Drawing List

The list of all standard and typical detail drawings is given in Table 1-1:

Table 1-1: Standard and Typical Detail Drawing List

Dwg No.	Drawing Title
Standard Drawings	
S-001	Standard Asset ID Plaque and Bench Mark Tablet
S-002	Standard Culvert Asset ID Tag
S-003	Steel Pile Details
S-004	TL-4 Double Tube Type Bridgerail, Bridgerail Details
S-005	TL-4 Double Tube Type Bridgerail, Approach Rail Transition Details
S-006	Barrier and Curb Details
S-007	TL-2 Thrie Beam Bridgerail
S-101	Prestressed Concrete 10m Type SL-510 Interior Girder
S-102	Prestressed Concrete 10m Type SL-510 Interior and Exterior Girder Layouts
S-103	Prestressed Concrete 12m Type SL-510 Interior Girder
S-104	Prestressed Concrete 12m Type SL-510 Interior and Exterior Girder Layouts
S-105	Prestressed Concrete 14m Type SL-510 Interior Girder
S-106	Prestressed Concrete 14m Type SL-510 Interior and Exterior Girder Layouts
S-107	Prestressed Concrete Type SL-510 Girder Standard Details
S-108	Type SL-510 Prestressed Concrete Bridges with Steel Substructures – Sheet 1
S-109	Type SL-510 Prestressed Concrete Bridges with Steel Substructures – Sheet 2
S-110	Type SL-510 Prestressed Concrete Bridges with Steel Substructures – Sheet 3
S-111	Type SL-510 Prestressed Concrete Bridges with Steel Substructures – Sheet 4
Typical Detail Drawings	
T-001	Installation of CMP and SPCMP Structures
T-002	Expansion Bearings
T-003	Steel Plate Girder Bridges – Sheet 1
T-004	Steel Plate Girder Bridges – Sheet 2

2. LIST OF DEFINITIONS AND ACRONYMS

2.1 Definitions

Definitions used in the Bridge Structures Design Criteria are in accordance with those provided in the CHBDC, unless otherwise noted below:

Approval (or Approved):	approval, or approved, in writing by the Department's Manager, Structures – Bridges.
Approved:	when referencing a product or material, approved means products or materials listed as "Approved products" on the Department's Product List.
Consultant:	the design consultant retained by the Department.
CHBDC:	the CSA S6:19 Canadian Highway Bridge Design Code.
Department:	the Government of Northwest Territories Department of Infrastructure
Detailed Design Drawings:	any drawings prepared by the Department's Consultant to be included in the construction tender package. These drawings must be prepared in accordance with the Department's Engineering Drafting Guidelines for Bridge Projects.
Professional Engineer:	a Professional Engineer registered with the Northwest Territories Association of Professional Engineers and Geoscientists (NAPEG)
Proprietary Structure:	proprietary structures are structures that are designed and built by the Contractor under a construction contract. Examples of proprietary structures are culverts, GRS/MSE walls (except for global stability), and sign structures.
Special Provisions:	the project specific construction specifications relating to material specification, construction methodology, quality testing requirements and payment which are prepared by or on behalf of the Department and are applicable to the Department's construction projects.
Supplier:	the supplier of a proprietary structure. Suppliers will typically be engaged by the Contractor under the Contract.
Slab-on-girder Bridge:	definition as per CHBDC Clause 5.2 (Definitions) and considered equivalent to a Deck-on-girder bridge as defined in CHBDC Clause 5.5.4 (Deck-on-girder).

2.2 Acronyms

The following acronyms apply to the Bridge Structures Design Criteria document:

AADT:	Average Annual Daily Traffic
AASHTO:	American Association of State Highway and Transportation Officials (USA)
ACP:	Asphalt Concrete Pavement
AISI:	American Iron and Steel Institute (USA)
AMPP:	Association for Materials Protection and Performance (USA)
ASTM:	ASTM International (formerly American Society for Testing and Materials) (USA)
AT:	Alberta Transportation (now Alberta Transportation and Economic Corridors (TEC))

AWS:	American Welding Society (USA)
BIM:	Alberta Transportation (TEC) Bridge Inspection and Maintenance
BPG:	Alberta Transportation (TEC) Bridge Best Practice Guideline
BSDC:	Alberta Transportation (TEC) Bridge Structures Design Criteria
CGS:	Canadian Geotechnical Society
CHBDC:	Canadian Highway Bridge Design Code, CSA S6
CSA:	Canadian Standards Association
CRR:	Corrosion Resistant Reinforcing
CSE:	Copper Sulphate Electrode
FHWA:	Federal Highway Administration (USA)
FLS:	Fatigue Limit States (see Clause 3.4 of the CHBDC)
GBH:	Guide to Bridge Hydraulics (TAC)
GRS:	Geosynthetic Reinforced Soil
HPC:	High Performance Concrete
IFC:	Issued for Construction
IFT:	Issued for Tender
LRFD:	Load and Resistance Factor Design
MOTT:	British Columbia Ministry of Transportation and Transit
MSE:	Mechanically Stabilized Earth
NACE:	National Association of Corrosion Engineers (USA)
NAPEG:	Northwest Territories Association of Professional Engineers and Geoscientists
NBCC:	National Building Code of Canada
NCHRP:	National Cooperative Highway Research Program (USA)
NSBA:	National Steel Bridge Alliance (USA)
PDA:	Pile Driving Analysis (see Clause 3.5.2 of the SSBC)
PSS	Procurement Shared Services
PTFE:	Polytetrafluoroethylene (also known by the brand name Teflon©)
PTI:	Post Tensioning Institute (USA)
PVC:	Polyvinyl Chloride
RDG:	Alberta Transportation (TEC) Roadside Design Guide
RWIS:	Road Weather Information System
SBDG*:*	GNWT Structures (Bridges) Design Guidelines (this document)
SLS:	Serviceability Limit States (see Clause 3.4 of the CHBDC)
SSBC:	GNWT Standard Specifications for Bridge Construction
SSHC:	GNWT Standard Specifications for Highway Construction
SSPC:	Society for Protective Coatings (USA)
TAC:	Transportation Association of Canada
TEC:	Alberta Transportation and Economic Corridors (formerly Alberta Transportation)
ULS:	Ultimate Limit States (see Clause 3.4 of the CHBDC)
UNS:	Unified Numbering System
WEAP:	Wave Equation Analysis of Piles

3. DESIGN LOADS

3.1 Highway Bridge Loads

Highway bridges are defined as all bridges carrying vehicular traffic with or without pedestrian traffic.

3.1.1 Dead Load

Dead loads shall include an allowance for an additional 50 mm concrete overlay over the full area of the bridge deck to account for future deck rehabilitation and also to partially account for any unanticipated dead loads that may be added to the structure following construction.

3.1.2 Live Load

For highway bridges, the design vehicle shall be a CL-800 Truck as defined in the CHBDC. No adjustments are required for the 9 kN/m uniformly distributed load for lane loads.

Live load distribution factors used for girder design shall not be less than the factors determined by the empirical method in accordance with Clause 5 (Methods of analysis) of the CHBDC unless otherwise Approved. If a bridge does not satisfy the criteria that allows the empirical factors to be used, the live load distribution factors used for girder design shall not be less than the empirical factors that would have been used if the bridge had met these criteria. The distribution factors used shall be shown on the Detailed Design Drawings.

As it relates to Clause 3.4.4 (Serviceability limit states) of the CHBDC, the anticipated degree of pedestrian use for all highway bridges with sidewalks shall be “occasional pedestrian use”. When a higher anticipated degree of pedestrian/cyclist use is identified, a request to use “frequent pedestrian use” shall be submitted for Approval.

Notwithstanding Clause 3.8.9 (Pedestrian load) of the CHBDC, pedestrian loads shall be determined in accordance with Clause 6.5 (Pedestrian Load) of CSA S7:23 (Pedestrian, cycling, and multiuse bridge design guideline).

3.1.3 Permit Vehicles

Unless otherwise noted, the Department requires that new bridges be designed to accommodate the special permit vehicles provided in Appendix A Permit Vehicle Configurations.

Load factors for these vehicles shall be determined in accordance with Section 14 (Evaluation) of the CHBDC, assuming inspection level INSP1.

3.1.4 Fatigue Design

All new highway bridges shall be designed to comply with Class A Highway requirements (Clause 1.4.2.2 (Highway class) of the CHBDC) regardless of traffic volumes. Fatigue stress ranges shall be determined using the CL-800 Truck. In addition, the C_L factor shall always be taken equal to 1.0 in Clause 10.17.2.2 (Design criteria) of the CHBDC. These requirements shall apply to all bridge components for considerations of structural fatigue.

All light poles mounted on bridges shall be designed to the requirements of AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals - Fatigue Category I.

3.2 Pedestrian Bridge Loads

Pedestrian bridges are defined as all bridges that do not carry motorized vehicular traffic, but instead carry pedestrian traffic and/or bicycle traffic and maintenance vehicles. The clear deck width of pedestrian bridges shall at a minimum meet that of the approach pathway, plus any additional width required for barrier shy offset as defined by the TAC Geometric Design Guide for Canadian Roads.

Guidance on pedestrian bridges is given in CSA S7:23 (Pedestrian, Cycling, And Multiuse Bridge Design Guideline). For pedestrian bridges, Consultants shall use this reference in conjunction with CHBDC and these guidelines.

For bridges that are expected to have high numbers of pedestrians or bridges on which pedestrians might reasonably be expected to stop and congregate, it may be more appropriate to design the pedestrian barrier in accordance with the loads and geometry requirements specified in the current National Building Code of Canada (NBCC). In this case, the selected barrier requires Approval.

3.3 Seismic Design Loads

Although seismicity is generally low in the Northwest Territories, thus placing the majority of bridge designs into Seismic Performance Category 1, there are regions of moderate to high seismicity which trigger the use of Seismic Performance Category 2 or 3 - particularly in the west of the Northwest Territories, between the Mackenzie River and the Yukon border. These higher performance categories require more rigorous analysis methods and design details to be incorporated into the design and may also require greater geotechnical investigation effort.

Note that although Seismic Performance Category 1 structures do not require seismic analysis, seismic design requirements still apply – such as horizontal restraint force and minimum seat length requirements (CHBDC Clauses 4.4.10.2 (Seismic performance category 1) and 4.4.10.5 (Minimum support length requirements for displacements)).

When applying Clause 4 (Seismic design) of the CHBDC, all bridges located on a numbered highway shall be considered “Major-route bridges”. All other bridges shall be considered “Other bridges”.

Seismic design shall be conducted using the Earthquakes Canada Seismic Hazard Tool (https://earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index-en.php) established for the NBCC as adopted by the CHBDC. The latest edition shall be used unless otherwise Approved. When the seismic hazard edition changes during a project, the designer shall present a review of the new and previous NBCC hazard and discuss with the Department if updates to the design are required.

Note that beginning with the 2020 NBCC, the online seismic hazard tool now accounts for the geotechnical Site Class when determining the seismic hazard values. As a result, the site coefficient adjustments outlined in Clause 4.4.3.3 (Site properties) of the CHBDC and tabulated in Tables 4.2 through 4.9 of the CHBDC are no longer required.

3.4 Load Evaluation and Bridge Rehabilitation

Evaluation of existing bridges shall be in accordance with Section 14 (Evaluation) of the CHBDC.

Design for bridge rehabilitation is normally based on meeting CHBDC Section 15 (Rehabilitation and repair). It should be recognized that when bridge strengthening is required to meet the CL1, CL2 and CL3 vehicles from the CHBDC, it may be possible to increase the strengthening design to meet the CL800 design vehicle with minimal impact on the overall rehabilitation cost. Considerations should include the type and age of the structure, expected remaining life, and expected use of the highway. The design vehicle to be used for rehabilitation shall be Approved before commencing the design.

Note that Section 15 (Load evaluation) of the SSBC provides requirements for load evaluation by the Contractor and should be referenced and complemented where needed. Load rating of the substructure is not explicitly discussed in the SSBC, and requirements would need to be included in the Special Provisions if needed.

3.5 Environmental Loads

In general, design for environmental loads such as temperature, ice, wind, etc. shall follow the requirements of the CHBDC. The impacts of climate change may be causing changes to several environmental loads in the CHBDC in a manner not well understood. Examples could include thicker or thinner design ice, increased stream flows, or more variable wind loads on structures. When the bridge is governed by or sensitive to environmental loads, additional effort to ensure the potential variability in these loads over the life of the bridge shall be considered and shall be discussed with the Department for Approval.

Environment Canada Meteorological Stations are spaced far apart in the Northwest Territories. Designs should be typical to the closest Environment Canada Meteorological Station. If there is no station close enough to adequately characterize the climate on site, a site-specific study shall be done after discussion and Approval.

3.6 Construction Loads

Expected construction loads (such as overhang brackets and screed machines) shall be considered in the design and shown on the Design Drawings.

4. BRIDGE GEOMETRY

4.1 Hydrotechnical

4.1.1 Design Considerations

Hydrotechnical design for bridges, large culverts (greater than 1.5 m span), and channel control works shall be done in accordance with the procedures outlined in the following publications. In all cases, the most recent edition shall be used.

- TAC Guide to Bridge Hydraulics (GBH)
- CHBDC
- FHWA HEC-18 Evaluating Scour at Bridges (HEC18)
- FHWA HEC-20 Stream Stability at Highway Structures (HEC20)
- FHWA HEC-23 Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance (HEC23)

The Consultant shall present the hydrotechnical analysis and design methods, and results in a hydrotechnical design brief for the structure. This design brief must include all assumptions, survey plots, discussion of data sources, hydrologic analysis (including climate change assumptions), hydraulic model approach and results. Any special conditions for the bridge site, such as backwater effects and/or ice jams, must be identified and discussed. All potential options for the structure and all design recommendations must be included.

Bridges shall be designed to pass a 1% Annual Exceedance Probability (1% AEP or 100-year return period) discharge with the minimum freeboard specified below. The design discharge shall be adjusted to account for climate change impacts. Hydrological analysis requirements are discussed in Section 4.1.2 (Hydrological Analysis).

Design water surface elevations shall be based on the design discharge as determined by the hydrotechnical analysis, which shall include an assessment of the backwater conditions from downstream watercourses or waterbodies such as the Mackenzie/Deh Cho River and Great Slave Lake.

The minimum freeboard for free span bridges shall be 1.5 m over the design water surface elevation. This freeboard allowance includes a safety factor to address uncertainty in design discharge estimates and allows for the safe passage of floating debris. On watercourses where floating debris is unlikely, the Department may relax this requirement based on the recommendation of the hydrotechnical design engineer, in which case the minimum freeboard shall not be less than 1.0 m. The Consultant shall obtain Approval for such relaxation before proceeding with the design.

Where bridge-size culverts (1.5 m span up to 6 m span) are considered, they shall have a minimum freeboard of 0.3 m between the design water surface elevation and the crown of the culvert. If significant floating debris is possible, a bridge with a 1.5 m freeboard allowance is preferable to a culvert. Culverts under outlet control shall have a total head loss over the length of the culvert not exceeding 0.3 m plus 0.1 m per 10 m of culvert length. This head loss constraint includes the culvert entry and exit losses.

The hydrotechnical analysis shall include consideration of aspects such as channel degradation and aggradation, lateral stability of the channel, potential for interruption of bedload movement through the structure and other similar geomorphological concerns.

Fish passage requirements shall be evaluated and met through the hydrotechnical analysis and design process.

4.1.2 Hydrological Analysis

4.1.2.1 Base or Current Conditions

The design flow for the structure shall be determined using one of the following approaches (ranked in order of preference, subject to data availability):

- Intra-basin transfer of Flood Frequency Analysis results from a gauge on the same watercourse.
- Based on regional analysis of available gauges located in similar topography, vegetation, and climate.
- Hydrological modelling of the watershed (HEC-HMS, RAVEN or similar), considering the use of typical values or calibrating parameters with similar watersheds that are gauged.
- Rainfall/runoff analysis, using PCSWMM or similar software.
- For basins less than 20 km^2 only, Rational Method.

The hydrological analysis shall also identify the dominant mechanism for generating the design flow event, whether it be snowmelt, rain on snow, a rainfall event, or other mechanisms, and whether this may be a determining factor in the methods used to estimate the design flow.

4.1.2.2 Climate Change Assessment

The design flow estimation methods identified above are dependent on historical climate and/or hydrometric data and assume climate stationarity, and do not account for climate change. Accordingly, the Consultant must estimate the potential amplification effects of climate change on design flow rates and include these effects in the design.

Four possible methods are listed below in order of preference. At least two of these methods shall be employed to allow comparison of the relative order of magnitude determined for the adjustment. The effort expended on this analysis shall be to a level appropriate to the importance of the structure and available data and budget.

- Use of hindcast and forecast climate model output data for an ensemble of climate models. The output data should be downscaled to a climate station proximate to the watershed. The downscaled climate data can be used with hydrologic modelling of the watershed as mentioned above, to estimate the proportional difference between historic and climate change design flows. This requires a significant level of effort and should be undertaken only when directed by the Department.
- Use of estimated impacts to climate norms (rainfall and snowpack) or other climate impact estimates as available from the Canada Climate Change Data Portal (http://canadaccdp.ca).
- Use of Clause 6.3.4.9 (Applying a simple "increase" factor using CC relation) of CSA Plus 4013:19 (Technical Guide: Development, interpretation, and use of rainfall intensity-duration-frequency (IDF) information: Guideline for Canadian water resources practitioners). The scaling method described in this standard is recommended for use by Environment and Climate Change Canada (ECCC). It follows the theoretical Clausius-Clapeyron relation of a 7% increase in atmospheric moisture per 1 °C increase in temperature.
- Use of the University of Western Ontario (UWO) IDF-CC tool for scaling precipitation (https://www.idf-cc-uwo.ca) using an ensemble of bias-corrected climate model outputs.

Estimation of climate change impacts should consider data and results derived from the Coupled Model Intercomparison Project Phase 5 (CMIP5) RCP4.5, RCP6.0 and RCP8.5 scenarios, or alternatively, the more recent Coupled Model Intercomparison Project Phase 6 (CMIP6) SSP2-4.5, SSP4-6.0 and SSP5-8.5 and scenarios as model output becomes more readily available.

Secondary climate change effects, including permafrost degradation and wildfire or other vegetation disturbances, shall be assessed and included in design flow estimates where relevant. In particular, wildfire impacts can be widespread in a watershed and remain relevant for decades before natural revegetation occurs.

Climate change is likely to result in widespread degradation of permafrost. Where permafrost is present within the watershed upstream of a proposed crossing, the potential impact of degradation of the permafrost shall be considered in the context of potential hydrological impacts, and morphological effects. This requires specialist input as the potential impacts are varied and not readily predicted.

4.1.3 Hydraulic Analysis and Design

4.1.3.1 General

The hydraulic analysis and design of a bridge and approaches require both a quantitative and qualitative effort. The Department expects the hydraulic analysis for proposed crossings to be undertaken by modelling in HEC-RAS or an equivalent software platform with an established record for accuracy and reliability. One-dimensional modelling is sufficient for most crossings. However, two-dimensional modelling should be used where a large crossing structure is anticipated, or the channel hydraulics are complex as directed by the Department.

Assuming that the required survey data is available or will be obtained, the watercourse reach for a minimum of 100 m upstream and downstream of the proposed crossing shall be included in the hydraulic model. Where larger/longer structures are proposed, or significant hydraulic features are present that could impact the proposed crossing, the model extents shall be increased appropriately. In particular, downstream conditions that may impart a backwater condition on the site (such as a lake or large river that influences flow conditions in the watercourse the structure is located on) must be addressed in the modelling.

Downstream influences may also be addressed by specifying a known boundary condition at the downstream extents of the model. These could address freshet levels in lakes and rivers, or flow impediments due to an ice jam or other structures located downstream of the proposed crossing. However, care should be taken in specifying these conditions to ensure a reasonable degree of confidence in the selected boundary condition. The hydraulic analysis shall follow these requirements:

- Hydraulic analysis shall be undertaken using the latest version of hydraulic modelling software, whether HEC-RAS or other.
- The model cross-sections and channel grade shall be based on topographic survey.
- Model hydraulic parameters shall be based on the interpretation of field observations, supplemented by general literature.
- When coinciding hydrometric data and field measurements of high water levels exist, the model shall be calibrated. All HEC-RAS model cross-sections showing normal and high water and/or ice levels shall be presented in appendices of the design brief.

- Determine HEC-RAS results for design flow rate (including climate change), and fish passage flow for:
  - existing structure (if applicable).
  - natural channel (no structure or roadway on the floodplain).
  - proposed configuration of the new structure, including erosion and scour protection, and any river training works.

The required culvert and bridge waterway opening dimensions shall be estimated considering the potential impacts of the structure on the following:

- Constriction of the natural channel and floodway.
- Disruption of natural sediment transport, with aggradation or local scour as a result.
- Introduction of backwater effects that may cause or worsen upstream flooding with impacts to natural habitat, infrastructure, or property.
- Impediment to fish passage during critical periods. Estimate velocities for flow conditions during fish passage windows, consider high and low extremes.
- Potential morphological impacts of the proposed structure.
- Navigable Waterways requirements.
- Determine design water and/or ice elevation, design discharge, backwater effects, average flow velocity, freeboard, and preliminary minimum soffit elevation.

4.1.3.2 Scour and Erosion Evaluation

The results of the hydraulic analysis shall be used to estimate the potential scour elevations and erosion conditions for the proposed crossing. These include local, contraction, abutment and pier scour as applicable to the proposed structure. The scour and erosion assessment should be based primarily on the methods contained in TAC GBH, FHWA HEC18 and FHWA HEC23 and should consist of the following:

- Evaluate stream bed stability for the proposed bridge waterway opening.
- Develop preliminary design of scour and erosion protection, and channel control works:
  - Bank armour, guide banks, spurs and/or apron design as required.
  - Per SSBC section 10: riprap size, gradation, and quality should be evaluated, as should alternative protection measures if applicable.
  - Riprap layer thickness, noting that the minimum thickness should be at least 1.5 times the nominal D50 of the riprap gradation.
  - Filter design.
  - Erosion protection termination details.
  - Protection height, slope, toe/end treatment.
  - Work points required; toe elevation must be specified.

- Prepare a preliminary plan and elevation sketch of the proposed bridge waterway opening and details/cross sections of armoring.

Note that hydraulic modelling results are invalid and additional modelling is required if the proposed erosion and scour protection works significantly alter the dimensions of the channel from those in the initial analysis or present a significant change in channel roughness (Manning's "n").

4.1.3.3 Sensitivity Analysis

A sensitivity analysis is required for all quantities that have a significant influence on hydraulic results such as channel roughness parameters, scour depth multipliers, natural channel characteristics, and design flow rates. Generally, a variation of ±20% shall be considered. The design of the structure and associated hydrotechnical works shall be able to accommodate this variation without potential failure, though normal factors of safety (e.g., freeboard, etc.) may be reduced. A reduction of normal factors of safety reduction requires Approval.

4.1.3.4 Morphological Overview Assessment

The Consultant shall complete an overview of potential morphological risks to the proposed crossing using available information (aerial imagery, flood records, site history, etc.). This overview must include existing conditions such as lateral instability of the channel, channel degradation and aggradation. The potential for these conditions to directly impact the structure, or for outflanking the structure should be considered. The interaction of natural morphological processes with the structure, or for the structure to worsen or interfere with those processes shall be considered. If identified, appropriate mitigation measures should be developed and implemented.

4.2 Bridge Alignment

Unless otherwise Approved, bridges shall be on tangent horizontal alignments. Curved bridges require extra design and detailing, and cost more for construction and maintenance. If a curved bridge is selected, curve effects on operational safety, deck drainage, maintenance, etc., shall be fully considered during all engineering phases.

Where practical, skew on bridges should be minimized. Skewed bridges require extra design and detailing, and cost more for construction and maintenance. Skew on bridges should be kept to a maximum of 15 degrees. Skew larger than 15 degrees requires Approval at the conceptual design phase.

Bridge decks, including approach slabs and roof slabs, are susceptible to preferential icing (the bridge is icy when the approach road is not). As such, braking and steering adjustments on the bridge deck should be avoided. Solutions include reducing the longitudinal gradient, reducing the superelevation (tangent alignment or large radius curve), providing a constant road cross section across the bridge (no spirals, no tapers), and avoiding intersections on or in close proximity to the bridge (to avoid the need to brake on the structure).

4.3 Grades and Crossfall

For bridge deck drainage purposes, the Department considers a longitudinal grade of 1% to be the minimum acceptable value and Approval is required to use a longitudinal grade of less than 1%. Wherever possible, the top of crest curves shall not be located on the bridge and shall be located beyond the end of the bridge approach slabs. Grades below 1% can result in standing pools of water on the deck and may potentially cause operational and safety issues on the structure due to the presence of ice patches and may require retrofit drains to be installed after construction. In addition, grades of less than 1% affect the sub-surface drainage (below the ACP, if present), which results in a reduced service life of the ACP overlay and deck and accelerated structure deterioration.

Bridge decks shall have a 2% crossfall away from the crown line unless the bridge structure is superelevated, in which case the minimum superelevation shall be 2%.

Bridges that have a resultant slope of 4% or greater anywhere due to the combined effects of roadway grade and crossfall and bridges that are located in areas where changes in traffic speed are required shall be designed with considerations for an appropriate preferential icing mitigation strategy.

4.4 Headslopes, Sideslopes and Approaches

The finished ground top of the fill headslope width shall be equal to the "outside of bridge structure to outside of bridge structure" width plus at least 2.0 m (at least 1.0 m on each side of the bridge, as measured perpendicular to the faces of the wing walls).

The width of fills beyond the bridge (i.e., on the approaches) shall be sufficient to meet approach rail/guardrail standard requirements and shall transition to meet the roadway design side slope. Unless otherwise governed by approach barrier transitions, the width transition from the top of headslope to the top of the roadway side slope shall taper at a rate of 30:1.

Corner transitions between the bridge headslope and the side slope shall use an elliptical curve at the elevation of the toe of the headslope.

Approach roadways to bridges shall be designed to meet roadside safety requirements such as appropriate approach guardrail length, curved corner guardrails for intersecting roadways, include barrier end cushions, and must consider the safety of pedestrians and cyclists.

4.5 Bridge Layout

4.5.1 Span Layout

The overall length of a bridge structure is defined by the width of the feature being crossed combined with the roadway profile, the depth of the selected structure type, and the selected headslope grades. Span layout refers to the combination of the number of spans and the length of each span used to make up the overall length.

4.5.1.1 Single-Span Bridges

Bridges consisting of only a single-span are simple and cost-effective as they do not require intermediate piers. Single-span alternatives are preferred from a regulatory standpoint, as work within the watercourse is minimized.

A single-span solution should always be considered first, with multi-span solutions considered only if the single-span alternative is found to be uneconomical or not feasible.

Some of the reasons that a single-span alternative may be uneconomical or not feasible include the following:

- The required girder length may exceed the length or weight that can be easily transported to the site, or the girder weight may exceed the capacity of available lifting equipment. Some sites will have specific transportation restrictions that need to be considered but, in general, girders that can be carried on a typical semi-trailer (maximum length 16 m, maximum vehicle weight 625 kN) should not need special consideration.

- For precast concrete girders, the available girders may define the upper limit for a single-span. Looking at the TEC standard precast girders, for example, the maximum single-span ranges from 14 m to 20 m, depending on the specific girder type being considered.
- The "per meter" installed cost for a girder – i.e., the cost including fabrication, transportation, and erection – generally increases with span length, due to such things as increased material quantities, larger vehicle, and crane requirements, etc. As a result, the cost of two shorter girders will generally be less than the cost of a single longer girder; if the total difference in cost exceeds the cost to construct the intermediate piers, then the single-span alternative is no longer the most economical.

4.5.1.2 Multi-Span Bridges

The span arrangement for multi-span bridges should consider the following factors:

- Intermediate piers should be located to avoid the deepest and fastest sections of the watercourse below. In practice, this often means that a three-span alternative is selected over a two-span alternative, as the three-span alternative allows the piers to be located at the two edges of the watercourse. Depending on the topography, unsymmetric two-span alternatives may offer similar benefits, with the watercourse located under a longer span to allow the single pier to be pushed onto the far edge of the watercourse.
- Keeping the number of different span lengths to a minimum may minimize the overall cost of the structure due to economies of scale – fewer shop drawings, fewer changes to form bulkhead positions, etc.
- If the structure is to be made continuous for flexure over the intermediate piers (see Section 4.5.2 (Continuity and Articulation) for further discussion) then the relative length of end spans vs. interior spans needs to be considered. An end span that is around 75% of the main span is ideal; longer end spans will govern the girder design (perhaps resulting in uneconomical girder designs for the interior spans) while shorter end spans may not be feasible due to uplift at the abutments.

4.5.2 Continuity and Articulation

4.5.2.1 Continuity

Continuity refers to whether or not the adjacent spans of a multi-span bridge are detailed to allow the transfer of moments between the two spans. A continuous structure allows the transfer of moments and is analyzed as a continuous beam while a discontinuous structure does not allow the transfer of moments and is analyzed as a series of independent simple spans.

The continuity of a structure can change during construction. Precast concrete girders are discontinuous when erected but may be made continuous later by the addition of a continuous deck slab, for example. A structure of this type is often referred to as being "continuous for live load", since only loads applied after completion of the continuous deck links (such as superimposed loads or live loads) are carried on the continuous structure. Multi-span steel girder bridges are usually continuous from the time of erection, as individual segments are bolted together at field splices. The designer must ensure that the analysis for all loads correctly accounts for the continuity in place at the time those loads are applied.

In general, continuity is encouraged and will result in a more economical and robust structure.

4.5.2.2 Articulation

Articulation refers to how longitudinal forces (such as braking forces for example) are resisted and how longitudinal expansion and contraction of the bridge superstructure are accommodated.

For multi-span structures, the superstructure may be separated into multiple segments by the inclusion of expansion joints at one or more intermediate pier locations. As the expansion joints allow these segments to move independently, each segment must individually satisfy the requirements for stability and movement as outlined in the sections below.

New structures shall not include intermediate joints unless Approved. Approval will not be provided unless the total length of the structure results in thermal movements that cannot be accommodated by a continuous superstructure. As expansion joints are locations where maintenance and deterioration are focused, it is good practice to minimize the number of joints, even if it means using a more complicated or expensive joint type.

4.5.2.2.1 Stability (Longitudinal Forces)

Within each segment, there must be a load path to resist longitudinal forces applied to the superstructure. This load path usually consists of one or more support locations with either a fixed bearing (i.e., a bearing that allows rotation but not translation) or with a direct connection between the superstructure and the substructure (pier or abutment). For integral or semi-integral structures (discussed in Section 4.5.2.4 (Integral and Semi-Integral Abutments)) the load path may instead consist of earth pressure at the abutments bearing against the end of the superstructure.

4.5.2.2.2 Movement (Expansion and Contraction)

Each segment must be free to expand or contract without developing excessive axial forces within the superstructure. At deck level, the movement is typically accommodated by expansion joints. At girder level, the movement is typically accommodated by expansion bearings that allow the superstructure to move relative to the substructure below. For integral or semi-integral abutments (discussed in Section 4.5.2.4) the movement at both deck and girder level is accommodated by compression of the approach fills.

4.5.2.3 Detailing

No matter how the continuity and articulation are arranged, bearings and joints must be designed for the appropriate movements, including both translation and rotation.

For multi-span bridges, girders are sometimes discontinuous - i.e., there is no provision for the transfer of moments or shear at the pier. Joints should be avoided at these locations because they add to both the initial cost of construction and to future maintenance and rehabilitation requirements and are generally a vulnerable component in structures. Instead, link slabs should be installed to allow for deck continuity without moment continuity. Link slabs put additional rotational demands on the deck slab in this area and require special detailing to control crack widths and ensure that the required rotational capacity is provided.

4.5.2.4 Integral and Semi-Integral Abutments

An integral abutment is one where the superstructure is connected directly to the substructure with no provision for relative movement in the longitudinal direction. Longitudinal movement of the superstructure causes the substructure to move and the end face of the superstructure to push against the approach fills. Large longitudinal forces can be developed if approach fills are frozen or otherwise very stiff, but these forces can be resisted by fill at the opposite end of the structure. Unbalanced longitudinal forces during construction can be minimized by ensuring that abutments are backfilled simultaneously and by avoiding excessive compaction effort.

A semi-integral abutment includes expansion bearings between the superstructure and the substructure. As a result of these bearings, longitudinal movement of the superstructure has less effect on the substructure but the effect on the approach fills is similar to the effects of fully integral abutments.

See Section 6.4.3.6 (Integral Abutments) for more information.

4.5.3 Deck Width

A bridge width that matches the road width provides continuity for drivers. Functional aspects of bridge width include achieving shy line offsets to barriers, allowing safe storage of stalled vehicles, extending consideration to cyclists/pedestrians and other users, accommodating snow storage, allowing proper deck drainage, and safely accommodating traffic during maintenance activities. The optimal bridge width will balance functionality, capital construction cost, and lifecycle performance.

For bridges on rural roadways, the bridge deck clear roadway width shall match the width of the approaching roadway (existing or future planned). For long bridges, shoulders on bridges may be reduced for economic reasons, but this should be reviewed at the preliminary engineering stage and must be submitted for Approval.

Urban roadways often incorporate curb and gutter, and do not have shoulders. For bridges located on such urban roadways, the clear bridge deck width shall include shoulders that provide the appropriate shy distances to the bridge barriers. The transition between the bridge clear deck width (with shoulders) to the narrower approach roadway width (with curb and gutter) shall occur off the bridge. Occasionally, additional shoulder width may be required to meet minimum sight lines.

Criteria in setting the optimal clear roadway width for bridges with two or more lanes:

- The minimum bridge width shall be 9 m unless otherwise Approved.
- The bridge width shall match the design roadway unless otherwise required by the Department.
- The maximum shoulder width shall be 3.5 m to reduce risk of high angle strikes with barriers and reduce the likelihood of the shoulder being used as an extra lane.

Other factors to consider include:

- Stopping sight distance on curved structures may increase required shoulder widths.
- Extending roadway tapers across the bridge is desirable from structural design, operational, and future flexibility perspectives (i.e., a rectangular structure constructed wider than necessary is preferred over a tapered structure).

- Widening a bridge to improve deck drainage for infrequent rainfall events is typically not encouraged from a cost-benefit perspective.

The use of single lane bridges requires Approval and will only be allowed on select highway corridors. If a single-lane bridge is proposed on a highway, the following conditions shall be met:

- The bridge width shall be 6.0 m curb to curb.
- The expected vehicle types must be considered (e.g., Industrial requirements, truck volumes).
- AADT < 400 vehicles per day for the lifespan of the structure.
- Sufficient sight distance to see oncoming traffic must be provided. On long single-lane bridges, designers may consider refuge bays or localized deck widening to allow oncoming vehicles to safely pass.
- The Consultant shall identify signage to be provided along with any other strategies to mitigate hazards.
- The operating speed should be a maximum of 50 km/hr to enable vehicles to yield and minimize the severity of potential collisions.
- The likelihood of adherence to speed limits (driver behavior) should be considered when determining elements such as road geometry and sight lines.
- Assessments of how future rehabilitation will be undertaken must be considered and discussed with the Department, including traffic accommodation. Traffic accommodation could consist of complete closure of the bridge with a detour or provision of a temporary crossing.

Bridge decks shall not incorporate longitudinal joints. The clearance between two nominally parallel bridges shall not be less than 3.0 m in order to discourage pedestrians from attempting to jump from one bridge to the other. In the case where the adjacent bridge is not owned by the Department (pedestrian, etc.) the minimum clear distance between two nominally parallel bridges shall not be less than 10.0 m.

Bridges that are known to require widening in the future shall not use post-tensioned concrete girders.

The Department will advise the Consultant if future widening needs to be considered.

4.5.4 Bearing Setting

Bearings shall be assumed to be centered at a temperature of -10 °C, unless a site-specific study or construction considerations suggest that the bearing should be centered at a different temperature.

For sliding expansion bearings, a bearing temperature setting chart shall be provided for positioning bearing components according to the girder temperature at the time of bearing setting.

For concrete girders, the bearing design and setting chart shall make allowance for girder shortening due to post-tensioning and long-term shrinkage and creep.

4.5.5 Span and Girder Lengths

4.5.5.1 Steel Girder and Cast-in-Place or Segmental Concrete Superstructures

Span lengths established from preliminary engineering requirements shall be rounded up to the nearest whole meter.

Span lengths shown on the 'General Layout' Detailed Design Drawing shall be measured at a fabrication temperature of +20˚ C, from centreline bearing to centreline bearing along the bottom flange for uniform depth girders, and along the top flange for tapered or haunched girders.

For fixed bearings of continuous steel girder bridges, the design shall be based on bearings being centered on girder bearing stiffeners at -10˚ C. However, girder erection can happen over a wide range of temperatures, which cannot be determined at the time of design. Since the bearings should stay centred on the bearing stiffeners, the structure must be designed for any eccentricity due to the shift of the bearings. The size of voids for grouting anchor rods should have sufficient room to accommodate girder length changes at the time of erection, in addition to normal construction tolerances.

4.5.5.2 Precast Concrete Girder Superstructures

Girder lengths established from preliminary engineering requirements shall be rounded up to the nearest whole meter.

Length of precast concrete girders is to be shown on the 'General Layout' Detailed Design Drawings together with pier diaphragm thicknesses between girder ends (if applicable), and distance from girder end to centreline bearing at abutments.

Precast girder lengths shall be measured along the bottom flange at a fabrication temperature of +20˚C. The precast supplier shall make appropriate allowance for prestress shortening, shrinkage and creep up to the time of girder erection.

The bearing design and setting chart shall make allowance for girder shortening due to post-tensioning and long-term shrinkage and creep.

4.5.5.3 Horizontally Curved Superstructures

For curved structures with steel girders the girders shall be curved to match the curvature of the bridge unless otherwise Approved.

For curved structures with equal girder lengths (parallel chords) within each span, span length shall be measured along girder lines as defined above.

For curved or flared bridges with variable girder lengths (either curved or chorded) within a span, span length shall be measured along a selected girder line near the center on the 'General Layout' Detailed Design Drawing, with a cross-reference to the detailed 'Girder Layout' Detailed Design Drawing showing complete geometry of all girders.

4.5.6 Substructure Stations

Station values for locating the centerline bearing of substructure elements shall be adjusted to account for the following:

- Length difference between gradeline profile and horizontal surveyed distance.
- Length difference due to thermal change between +20˚C (assumed fabrication temperature) and -10˚C (temperature at which bearings are to be centered).
- When span lengths are measured along the top flange: longitudinal shift due to off-plumb tilting of bearing stiffeners or control sections set perpendicular to the top flange.
- Difference between grid and ground distances or other surveying systems (a reference for more information on ground and grid coordinates is TEC Design Bulletin #34 – Grid-to-Ground Survey Application). Stations are in grid coordinates; span lengths and other dimensions are in ground coordinates.

5. DURABILITY AND MATERIALS

5.1 General

The durability of bridge structures is essential to the overall good management of the Northwest Territories bridge inventory. Durability relies on quality design, proper detailing, and selection of appropriate materials and construction practices. This section addresses material requirements as well as a number of design considerations that are related directly to durability. Other design and detailing considerations are integrated into other sections; construction requirements are addressed in the SSBC.

Water, in particular water contaminated with de-icing salts, is one of the major causes of deterioration of structures. Design considerations to minimize this deterioration include the following:

- Direct water away from the structure.
- Minimize deck joints.
- Prevent the ingress of water.
- Prevent ponding on any portion of the structure, particularly on the bridge deck.
- Design a robust drainage system.
- Avoid detailing locations where water can be trapped.

Further guidance on drainage can be found in Section 5.5 (Control of Bridge Deck Drainage) and Section 14 (Bridge Drainage).

Bridge components shall be designed to be durable over their entire design life, under the current and future climatic conditions of the Northwest Territories, except for specifically identified replaceable components on the Detailed Design Drawings and as Approved by the Department.

5.2 Design Life

The minimum design life of bridge structures shall be:

- Bridges (including bridge-size culverts): 75 years
- MSE walls: 100 years
- Overhead sign structures: 50 years

There may be situations where a longer or shorter design life may be appropriate, for example: a shorter design life could be considered for structures that are planned to be temporary; a longer design life could be considered for major structures where replacement would be costly.

The use of a design life other than what is stated above requires Approval.

5.3 Operations and Maintenance Manual

Unless otherwise agreed upon with the Department, the Consultant shall prepare an Operations and Maintenance Manual which includes all components/elements for each new design and for rehabilitation projects on existing structures that do not have such a manual.

As a minimum, each Manual shall include the following:

- Expected service life.
- Comprehensive list of routine maintenance tasks to be completed, including the recommended schedule for each task.
- Expected schedule for component rehabilitation and/or replacement.
- Detailed instructions that may be necessary for specific component rehabilitation and/or replacement, such as bearings.
- Inspection requirements other than routine inspections carried out by the Department.

5.4 Bridge Decks, Roof Slabs, Approach Slabs and Approach Roadway

All bridge decks, roof slabs and approach slabs shall consist of Class HPC concrete.

For bridges with an overall length of 200 m or less, the wearing surface shall be HPC concrete, with a 10 mm allowance for loss of thickness due to abrasion. Quality concrete placement and good curing are essential to obtain the desired durability and performance of HPC. Bare concrete decks are prone to scaling if they are not appropriately cured before the onset of cold temperatures and the application of de-icing salts. If concrete placement is taking place during cold temperatures, additional concrete curing measures, such as extending the curing period, extensive heating, hoarding, etc., may be required, and these should be discussed with the Department and specified in the Special Provisions. The concrete finish shall be a Class 6 finish in accordance with the SSBC.

For bridges with an overall length exceeding 200 m, bridge decks, roof slabs and approach slabs shall be protected with a waterproofing system. Exceptions to this requirement may be appropriate for structures located along remote highways and where the mobilization of an asphalt plant will be cost prohibitive. The Department will confirm the waterproofing requirements for bridges with an overall length exceeding 200 m.

Where required, the waterproofing system shall be as shown on TEC standard drawings *S-1838, S-1839* and *S-1840 (Standard Waterproofing System for Deck and Abutments – Sheet 1, Sheet 2, and Sheet 3)*. The following requirements shall apply:

- The standard deck protection and wearing surface system shall have a total thickness of 90 mm consisting of a nominal 5 mm thick rubberized asphalt waterproofing membrane, plus 3 mm protective board and two 40 mm lifts of asphalt concrete pavement (ACP).
- ACP mix design requirements shall be determined by an engineer experienced in asphalt concrete pavement mix designs, and specific asphalt type, mix design and placement requirements to complement the SSBC shall be given in the Special Provisions.
- Waterproofing membranes shall include wick drains along the gutter lines to allow for controlled drainage of water that penetrates the ACP, and discharge of this water at the ends of the bridge only.

Other wearing surface types such as ultra-high performance concrete (UHPC) or polyester polymer concrete (PPC) may be considered by the Department, subject to Approval.

For bridges with an ACP waterproofing system, the approach roadways shall be paved for a minimum distance of 500 m on each side of the bridge, unless otherwise Approved. Paving of the approach roadways may also be considered on other bridge structures as a means to control damage to expansion joints and other elements of the structure by reducing the volume of gravel or chip seal material carried onto the bridge by traffic.

5.5 Control of Bridge Deck Drainage

As stated in Section 4.5.2 (Continuity and Articulation), the number of deck joints shall be kept to a minimum. Bridge decks are typically continuous over piers and only have joints at abutments, unless integral or semi-integral systems are used. All deck joints shall include provisions to capture and manage deck drainage such that it does not come into contact with other concrete and steel surfaces of other bridge elements other than dedicated drainage channels on the head and side slopes or drain troughs.

Joints around abutments and approach slabs shall be sealed at the surface and kept sealed throughout the life of the structure with proper maintenance. Any steel elements (including buried elements) that may potentially be exposed to leakage of chloride contaminated moisture shall be protected by an Approved impervious waterproofing membrane. The durability of the joint seal and means to protect the seal shall be considered in the design.

Refer to Section 14 (Bridge Drainage) for further guidance regarding drainage.

5.6 Splash Zone Surfaces

Splash Zone Surfaces are defined as surfaces subject to or potentially subject to roadway splash or spray and as a minimum shall include the following:

- All exposed faces of concrete curbs or barriers.
- Top surfaces of all pier and abutment concrete that project beyond the footprint of the bridge deck or bridge abutment, to a horizontal distance of 6 m from the inside edge of the bridge barrier/curb.

As indicated in Sections 5.8.1 (Concrete Classes) and 5.8.2 (Reinforcing Steel), if a particular bridge component includes a Splash Zone Surface, the entire component shall be constructed using the same upgraded concrete and reinforcing materials as required to satisfy the Splash Zone Surface requirements. If this requirement results in a significant increase in the quantity of HPC and/or CRR, the Department may consider Approving a reduction in the extent to which the upgraded materials are required.

Concrete drain troughs shall not be treated as splash zone surfaces.

5.7 Sealer

Concrete sealers shall be applied in accordance with the SSBC.

The use of Class 3 bonded surface finish, if applicable, shall be defined in the Special Provisions. Where pigmented sealers are used, the Consultant shall specify the colour scheme in the Special Provisions.

5.8 Materials

All materials selected for a design shall be specified in the notes of the appropriate Detailed Design Drawings, complete with the appropriate material properties used for the design (i.e., 28-day strength for concrete, yield and/or ultimate strength for steel, etc.).

5.8.1 Concrete Classes

Detailed information regarding concrete class material properties can be found in the SSBC. Note that CSA S6:19 and CSA S6:25 both refer to the 2019 version of CSA A23.1/2 which has seen significant changes between the 2019 and 2024 versions; hence, the Consultant should clearly indicate the version of CSA A23.1/2 is to be used on the drawings. Unless otherwise specified by the Department, the Consultant shall use the 2019 version of CSA A23.1/2 for their designs.

Table 5-1: Concrete Classes defines which concrete class shall be used for each bridge component.

Table 5-1: Concrete Classes

COMPONENT DESCRIPTION	CLASS	f'_c @ 28 days
Precast bridge girders. Refer to SSBC Section 7 (Precast concrete units).		50 to 70 MPa
Concrete decks, curbs, barriers, sidewalks, and medians; cast-in-place or precast. Cast-in-place concrete deck overlays. Abutment & pier diaphragms. Deck joint blockouts. Abutment roof slabs, approach slabs and sleeper slabs. *For abutment backwalls that support approach slabs or roof slabs on ledges/corbels, a construction joint shall be provided in the backwall below the ledge/corbel. All concrete above the construction joint shall be included in this category.* Precast concrete partial depth deck panels. MSE wall precast concrete fascia panels and MSE wall cast-in-place copings. All components containing one or more Splash Zone Surfaces. *Where any portion of the component is a Splash Zone Surface, the entire concrete component shall be included in this category.* Girder shear keys (14 mm max. aggregates).	HPC	45 MPa

COMPONENT DESCRIPTION	CLASS	*f'c* @ 28 days
Pile caps. Substructure elements (bearing seat, wing walls, footings, pier shafts, pier caps, pile caps, etc.) and monolithic concrete protection barriers other than those specified as Class HPC. Walls of concrete culverts/tunnels, unless considered a Splash Zone Surface. Overhead sign structure foundations. Drilled cast-in-place concrete piles above the frost line. Concrete drain troughs. MSE wall levelling pads. All other concrete elements not specified elsewhere in this Table.	C	35 MPa
Pipe pile infill concrete. Drilled cast-in-place concrete piles below frost line. *For piles that are expected to have dry holes.*	Pile	30 MPa
Pipe pile infill concrete. Drilled cast-in-place concrete piles below frost line. *For piles that have a reasonable risk of not having a dry hole.* Any other concrete that is to be placed under water.	Tremie	30 MPa

Notwithstanding the concrete classes identified in Table 5-1: Concrete Classes, where the minimum dimension of the concrete component is greater than or equal to 2.0 m, the Consultant shall specify mass concrete for the component, in accordance with the SSBC.

All concrete specifications in this document and the SSBC are based on Type GU or Type GUL cement. Other types of cement require specific considerations for mix designs and performance in terms of strength and durability. Hence, the use of other cement types requires Approval and additional specifications in the Special Provisions in construction contracts. Consultation with a materials engineer experienced in concrete materials and mix designs is advised. Sulphate testing shall be done for native materials and water near the concrete element for which Type GUL cement is used or at locations where sulphate content could be an issue for concrete durability.

Where concrete is applied in restrained conditions, for example as an overlay, the Consultant may consider specifying shrinkage tests and limits for the concrete. It should be noted that shrinkage tests require time – the minimum test duration for shrinkage tests in accordance with CSA A23.2-21C is 28 days, but generally results are more meaningful if measurements are taken up to 90 days. However, the results can be useful to get an indication of cracking and durability in certain applications. Other material-related measures that may help to reduce cracking of restrained concrete elements include specifying certain parameters in the mix design such as limiting the fine contents and having a mix that gains strength more slowly. Higher concrete strength often does not improve cracking behaviour and durability. Moreover, good curing, preferably 7-day wet curing in appropriate temperatures, significantly increases the durability of concrete components, and no shortcuts or relaxation of curing requirements should be allowed.

Concretes in regions with sulphates require special mix designs to achieve adequate durability, sometimes including the need for High Sulphate Resistant Type HS or HSb cement. While there are few locations in the Northwest Territories with sulphates, it is recommended that sulphate testing be carried out as part of the geotechnical investigation, in particular, if Type GUL cement may be used on the project.

5.8.2 Reinforcing Steel

5.8.2.1 Types of Reinforcing Steel

Reinforcing steel shall either be carbon (black, plain) steel or Corrosion Resistant Reinforcing (CRR).

CRR materials shall be one of the following types:

- Solid stainless reinforcing.
- Low carbon/chromium reinforcing (such as MMFX/ChromX).

The use of other reinforcing types (such as Fiber Reinforced Polymer (FRP)) should be discussed during design development and requires Approval. Use of other reinforcing types will require additional Special Provisions in construction contracts.

Reinforcing steel shall conform to the grades outlined in Table 5-2: Reinforcing Steel Types. Type A, B and C are used to distinguish different levels for CRR protection.

Table 5-2: Reinforcing Steel Types

REINFORCING TYPE	SPECIFICATION/DESIGN YIELD
Type A (Stainless steel)	Alloy types 316LN, Duplex 2205 or Duplex 2304 (Unified Numbering System (UNS) designations S31653, S31803 or S32304) meeting the requirements of ASTM A276/A276M and ASTM A955/A955M including Annex 1.2 or 1.3 with a minimum yield strength of 420 MPa. *The design and proportioning of the stainless reinforcing steel, including hooks, development lengths and bar splices, shall be based on a yield strength of 420 MPa.*
Type B (Stainless steel)	Alloy type XM-28 (UNS designation S24100) meeting the requirements of ASTM A276/A276M and ASTM A955/A955M including Annex 1.2 or 1.3 with a minimum yield strength of 420 MPa. *The design and proportioning of the stainless reinforcing steel, including hooks, development lengths and bar splices, shall be based on a yield strength of 420 MPa unless otherwise Approved.* Note that the lower nickel content of Type B may provide cost savings over Type A without noticeably affecting corrosion resistance in many applications. When stainless steel is required for bridge applications, Type B is usually sufficient; requiring the use of only Type A requires Approval.

REINFORCING TYPE	SPECIFICATION/DESIGN YIELD
Type C (Low carbon/chromium)	ASTM A1035/A1035M alloy type CS with a minimum yield strength of 690 MPa based on the 0.2% offset method. *The design and proportioning of the low carbon/chromium steel reinforcing bar, including hooks, development lengths and bar splices, shall be based on a yield strength of 500 MPa.*
Carbon steel (plain, black)	CSA G30.18 Grade 400W or 500W with a minimum yield strength of 400 MPa or 500 MPa. *CSA G30.18 Grade 400R or 500R shall not be used without Approval.* *Where Approved, CSA G30.18 Grade 400R or 500R shall not be used on the same project where Grade 400W or 500W is used.*
Deformed welded wire reinforcement	ASTM A1064/A1064M Grade 485 with a minimum yield strength of 485 MPa based on the 0.2% offset method.

Type B CRR (XM-28) is a high manganese low nickel austenitic stainless steel, strengthened by extra nitrogen present in solid solution, and has lower nickel content compared to the Type A CRR.

Where the use of FRP reinforcing is Approved, the following requirements shall be followed:

- Only Grade III Glass Fiber Reinforced Polymer (GFRP) reinforcement shall be used.
- Nominal cross-sectional area values specified in CSA S807 shall be used in the design (note that the nominal area values do not correspond directly to the designated diameter – i.e., the nominal area of a bar cannot be calculated using the designated diameter).
- Bars with a designated diameter below 13 mm shall not be used, as the smaller bars are susceptible to physical damage during normal construction operations.
- Bars with a designated diameter greater than 25 mm shall not be used due to issues with performing required QC and QA testing.

5.8.2.2 Design Requirements

The design of bridge components containing CRR shall be based on the reinforcing steel having a yield strength of 420 MPa. This includes all hooks, development lengths and bar splices.

The design of stirrups or projecting girder stirrups with low carbon/chromium ASTM A1035 reinforcing may be based on the reinforcing steel having a yield strength of 500 MPa. This includes all hooks, development lengths and bar splices.

Currently, ASTM A1035 reinforcing is only produced in imperial bar sizes. Stainless steel reinforcing is available in both metric and imperial sizes. The design of bridge components shall consider the high probability that a Contractor will request a substitution of imperial bars for metric bars. Where substitutions will be accepted, the design should be based on the smaller bar area, as shown in Table 5-3. Any proposed substitution of metric for imperial bars must be accepted by the Department.

Table 5-3: Comparison of Metric and Imperial Bars (areas in square millimetres (mm^2) given in brackets)

METRIC SIZE	10M (100)	15M (200)	20M (300)	25M (500)	30M (700)		35M (1000)	45M (1500)
IMPERIAL SIZE	#4 (129)	#5 (200)	#6 (284)	#8 (509)	#9 (645)	#10 (819)	#11 (1006)	#14 (1452)

5.8.2.3 Detailing Considerations

Where only stainless steel is intended as CRR (i.e., Type A or B), bar marks shall use the "SS" suffix on the bar mark. Where CRR may be either stainless steel or low carbon/chromium reinforcing (i.e. Type A, B or C), bar marks shall use the "CR" suffix on the bar mark. Where only low carbon/stainless steel (i.e. Type C) is intended as CRR, bar marks shall use the "MX" suffix on the bar mark. Bar marks for GFRP reinforcing shall use the "G" prefix on the bar mark along with the nominal bar diameter. The suffix M is omitted from GFRP bar identifiers as the bar sizes do not conform to those of Canadian, metric steel reinforcement. Bar diameter designators and nominal bar areas are defined in CSA S807.

When preparing the record drawings, the reinforcing type must be indicated on the record drawings by updating all bars originally carrying the "CR" suffix to either "SS-A" or "SS-B" for Type A or Type B stainless steel reinforcing respectively, or "MX" for low carbon/chromium reinforcing (Type C).

Notes added to the record drawings are acceptable in place of revising individual marks only if the specific material(s) can be clearly identified without reference to specific marks. For example, notes like

> "ALL BAR MARKS INDICATED AS 'CR' WERE PROVIDED AS TYPE B STAINLESS STEEL" OR "ALL BAR MARKS INDICATED AS 'SS' WERE PROVIDED AS TYPE A STAINLESS STEEL" ARE ACCEPTABLE.

All hooks and bends shall be detailed using the pin diameters and dimensions recommended for "black reinforcing" in the Reinforcing Steel Institute of Canada (RSIC), Reinforcing Steel - Manual of Standard Practice.

Welding of structural reinforcing is prohibited. Welding of additional non-structural reinforcing shall be submitted for Approval. If Approved, Grade 400W or 500W reinforcing shall be used.

The minimum size of reinforcing bars (excluding welded wire mesh) in all bridge elements shall be 15M.

5.8.2.4 Use of CRR Materials

The increasing use of CRR materials has been driven mainly by jurisdictions with high traffic volumes and a history of significant use of de-icing salts. In the Northwest Territories, traffic volumes are lower and the use of de-icing salts is less prevalent (although the use of de-icing salts is increasing in some areas). It would be excessive for these guidelines to explicitly require the use of CRR in every structure, but the remoteness of many structures (leading to high rehabilitation costs) may justify the use of CRR.

Instead of defining prescriptive requirements, these guidelines identify three levels of CRR usage. The levels range from CRR1 (including only plain black reinforcing) to CRR3 (including CRR for all relevant elements).

The Department will identify, in conjunction with the Consultant, the level to be followed in the design of each specific structure. The level to be followed is at the sole discretion of the Department, but the decision may consider some of the following issues:

- Single-lane structures are difficult to rehabilitate while maintaining a reduced number of traffic lanes. Single-lane structures that cannot be closed without seriously impacting the transportation network may be considered for a higher level.
- Rehabilitation of structures with a higher traffic volume causes a greater disturbance to the public, and as a result may be considered for a higher level.
- Rehabilitation of structures with difficult access requirements is more expensive, and as a result may be considered for a higher level.

Table 5-4: Reinforcing Steel Type by Location for Bridges with Concrete Wearing Surface and Table 5-5: Reinforcing Steel Type by Location for Bridges with ACP Wearing Surface specify the reinforcing type that shall be used in the specified locations on bridges within each CRR level for bridges with concrete wearing surfaces and with ACP wearing surfaces respectively.

B = black reinforcing
CRR = corrosion-resistant reinforcing (Type A, B, or C)
SS = stainless steel (Type A or B only).
WWR = deformed welded wire reinforcement

Table 5-4: Reinforcing Steel Type by Location for Bridges with Concrete Wearing Surface

LOCATION/DESCRIPTION	REINFORCING TYPE		
	Level CRR1	**Level CRR2**	**Level CRR3**
Top mat of reinforcing in full-depth cast-in-place decks and partial-depth cast-in-place decks cast on precast concrete partial-depth deck panels.	B	CRR	SS
Bottom mat of reinforcing in full-depth cast-in-place decks and partial-depth cast-in-place decks cast on precast concrete partial-depth deck panels.	B	B	CRR
Reinforcing bars projecting from precast concrete partial-depth deck panels.	B	CRR	SS
Reinforcing in curbs and barriers above the deck or wing wall construction joint, including dowels projecting through the construction joint.	B	CRR	SS
Reinforcing in sidewalks.	B	CRR	SS

LOCATION/DESCRIPTION	REINFORCING TYPE		
	Level CRR1	**Level CRR2**	**Level CRR3**
Top mat of reinforcing in abutment roof slabs, approach slabs and sleeper slabs. *For abutment backwalls that support approach slabs or roof slabs on ledges/corbels, a construction joint shall be provided in the backwall below the ledge/corbel. All reinforcing located above the construction joint shall be included in this category.*	B	CRR	SS
Bottom mat of reinforcing in abutment roof slabs, approach slabs and sleeper slabs. *For abutment backwalls that support approach slabs or roof slabs on ledges/corbels, a construction joint shall be provided in the backwall below the ledge/corbel. All reinforcing located above the construction joint shall be included in this category.*	B	B	CRR
Reinforcing located within 300 mm of the upper portions of abutment diaphragms and wing walls.	B	CRR	SS
All Splash Zone Surfaces. *Where any portion of the component is a Splash Zone Surface, the entire component shall be included in this category.*	B	CRR	SS
Stirrups projecting from precast or cast-in-place concrete girders into deck slabs.	B	CRR	SS
Reinforcing in precast girders, except for stirrups projecting from girders into deck slabs.	B/WWR	B/WWR	CRR
Stirrups projecting from precast girders into deck slabs.	B	CRR	CRR
Reinforcing in deck joint blockouts. Reinforcing in corbels and dowels connecting approach slabs to corbels.	B	CRR	SS
Reinforcing in MSE wall precast concrete fascia panels.	B	CRR	CRR
Reinforcing in all locations not otherwise specified.	B	B	B

Table 5-5: Reinforcing Steel Type by Location for Bridges with ACP Wearing Surface

LOCATION/DESCRIPTION	REINFORCING TYPE		
	Level CRR1	Level CRR2	Level CRR3
Top mat of reinforcing in full-depth cast-in-place decks and partial-depth cast-in-place decks cast on precast concrete partial-depth deck panels.	B	CRR	CRR
Bottom mat of reinforcing in full-depth cast-in-place decks and partial-depth cast-in-place decks cast on precast concrete partial-depth deck panels.	B	B	CRR
Reinforcing bars projecting from precast concrete partial-depth deck panels.	B	CRR	CRR
Reinforcing in curbs and barriers above the deck or wing wall construction joint, including dowels projecting through the construction joint.	B	CRR	SS
Reinforcing in sidewalks.	B	CRR	SS
Top mat of reinforcing in abutment roof slabs, approach slabs and sleeper slabs, where ACP and waterproofing membranes are installed. *For abutment backwalls that support approach slabs or roof slabs on ledges/corbels, a construction joint shall be provided in the backwall below the ledge/corbel. All reinforcing located above the construction joint shall be included in this category.*	B	CRR	CRR
Bottom mat of reinforcing in abutment roof slabs, approach slabs and sleeper slabs. *For abutment backwalls that support approach slabs or roof slabs on ledges/corbels, a construction joint shall be provided in the backwall below the ledge/corbel. All reinforcing located above the construction joint shall be included in this category.*	B	B	CRR
Reinforcing located within 300 mm of the upper portions of abutment diaphragms and wing walls.	B	CRR	SS
All Splash Zone Surfaces. *Where any portion of the component is a Splash Zone Surface, the entire component shall be included in this category.*	B	CRR	SS
Stirrups projecting from precast or cast-in-place concrete girders into deck slabs.	B	CRR	CRR

LOCATION/DESCRIPTION	REINFORCING TYPE		
	Level CRR1	Level CRR2	Level CRR3
Reinforcing in precast girders, except for low carbon/chromium stirrups projecting from girders into deck slabs.	B/WWR	B/WWR	CRR
Reinforcing in deck joint blockouts. Reinforcing in corbels and dowels connecting approach slabs to corbels.	B	CRR	SS
Reinforcing in MSE wall precast concrete fascia panels.	B	CRR	CRR
Reinforcing in all locations not otherwise specified.	B	B	B

5.8.3 Prestressing Steel

For use in pretensioned and post-tensioned concrete, prestressing strands shall conform to ASTM A416/A416M Grade 1860, low relaxation strand with a minimum tensile strength of 1860 MPa.

Prestressing rods shall conform to ASTM A722/A722M with a minimum tensile strength of 1035 MPa. Note that prestressing rods have experienced brittle failure under some conditions; their use should be undertaken with caution and only if Approved.

For soil nail wall systems, the soil nails shall be PTI - Class 1, Double Corrosion Protection (DCP) system (PTI (2014)). They are typically galvanized, centered in a protective sheath consisting of corrugated high-density polyethylene (HDPE) or polyvinyl chloride (PVC) pipe, and the annulus filled with an Approved grout.

5.8.4 Structural Steel

Table 5-6: Structural Steel Grades specifies the structural steel grades and material properties to be used in the specified locations on bridges. Where more than one grade is provided, both are considered acceptable, and the Consultant shall select as required to meet the design requirements. Occasionally, contractors will request material substitutions that are essentially equivalent to those used for design. The more common rough equivalent grades are included in brackets below. The strengths of these equivalent grades may not be exactly the same and it is the Consultant's responsibility to ensure that the design intent is satisfied when approving the substitution.

Any structural steel components in addition to those listed in Clause 6.2 (Supply and fabrication) of the SSBC shall have their grades specified in the Special Provisions.

Table 5-6: Structural Steel Grades

COMPONENT DESCRIPTION	GRADES
Girders, floor beams, stringers and all material welded to these members, such as stiffeners.	CSA G40.21 Grade 350AT CAT 3 or better *(ASTM A709 Grade 345WT Type B with Charpy value of 27 J @ -30°C)*
Steel structure diaphragm and bracing members, and their associated connection plates, which are considered Primary components (even if bolted to the girders). *This typically applies to diaphragm and bracing members in heavily skewed bridges or bridges with, curved or kinked girders.*	CSA G40.21 Grade 350AT CAT 2 or better *(ASTM A709 Grade 345WT Type B)* *(ASTM A588/A588M Grade 50 (345) Grade B with Charpy value of 27 J @ -20°C)*
Steel structure diaphragm and bracing members, and their associated connection plates, which are considered a Secondary component and bolted to the girders (connection plates that are welded to the girders fall under Item 1). Ungalvanized bearing materials.	CSA G40.21 Grade 350A *(ASTM A709 Grade 345W Type B)* *(ASTM A588/A588M Grade 50 (345) Grade B)*
Galvanized bearing sole plates and galvanized rocker plates, that are bolted to steel girders. Galvanized shoe plates cast into precast concrete girders and galvanized sole plates welded to shoe plates. Galvanized bearing base plates. Galvanized precast concrete girder bracing members which are bolted to the girders.	CSA G40.21 Grade 300W or Grade 350W *(ASTM A572/572M Grade 50 (345)*
Miscellaneous galvanized steel components.	CSA G40.21 Grade 300W *(ASTM A36)*
Galvanized steel bridge rail components – structural tubing.	ASTM A500 Grade B *(F_y = 315 MPa and F_u = 400 MPa)* ASTM A500 Grade C *(F_y = 345 MPa and F_u = 425 MPa).*
Galvanized steel bridge rail components – plate and other structural shapes.	CSA G40.21 Grade 350W *(ASTM A36)*
Shear studs.	Conform to the chemical requirements of ASTM A108 Grades 1015, 1018, or 1020 and meet the mechanical properties specified in AASHTO/AWS D1.5M/D1.5, Table 7.1 for Type B studs.
Structural steel bolts for weathering steel structural applications.	ASTM F3125 Grade A325M Type 3 weathering

COMPONENT DESCRIPTION	GRADES
Structural bolts for galvanized steel applications.	ASTM F3125 Grade A325M Type 1, galvanized. Galvanized direct tension indicator washer in accordance with ASTM F959.

5.8.5 Corrugated Metal

Corrugated metal for use in Corrugated Metal Pipe (CMP) and Structural Plate Corrugated Metal Pipe (SPCMP) culvert structures shall be in accordance with relevant standards. Refer to Section 18 of the SSBC.

5.8.6 Anchor Rods

Table 5-7: Anchor Rod Materials specifies the materials to be used in the specified locations on bridges.

Table 5-7: Anchor Rod Materials

DESCRIPTION	GRADES
Stainless steel bearing anchor rods in contact with black steel.	Stainless steel AISI Type 316 *(F_y = 290 MPa based on 0.2% offset)*
Galvanized steel bearing anchor rods in contact with galvanized steel bearing plates.	CSA G40.21 Grade 300W or ASTM A307, galvanized in accordance with ASTM F2329
Galvanized high strength steel anchor rods in contact with galvanized steel base plates (e.g., galvanized bridge rail post base plates).	ASTM A193 Grade B7 *(F_y = 725 MPa and F_u = 860 MPa).* *Note that galvanizing of high strength steel anchor rods requires a modified procedure in accordance with SSBC Section 6 (Structural steel) and SSBC Section 12 (Bridgerail).*

5.8.7 Steel H-Piles

Steel H-piles shall meet the requirements of CSA G40.21 Grade 350W or A572/A572M Grade 50 (345). Steel H-piles designed to be exposed (e.g., pier columns) shall be galvanized to a minimum depth of 1000 mm below ground or stream bed, in accordance with ASTM A123/A123M.

Whenever possible steel H-piles should be selected from the sizes shown on *S-020 (Steel Pile Details)*.

5.8.8 CMP and SPCMP

CMP or SPCMP shall be either aluminum or steel pipe. All pipe materials and markings shall be in accordance with CSA G401.

For aluminum, CMP or SPCMP materials shall conform to ASTM B745 or ASTM B746. Aluminum bolts shall meet the requirements of ASTM F468. Aluminum nuts shall meet the requirements of ASTM F467. All nuts and bolts shall be made from alloy 6061-T6.

5.9 Clear Concrete Cover

The clear concrete cover for reinforcing steel, pretensioning strands, and post-tensioning ducts shall be in accordance with the CHBDC, including requirements for abrasion and harsh environments, with the exceptions shown in Table 5-8: Clear Concrete Cover for Cast-in-Place Concrete Components and Table 5-9: Clear Concrete Cover for Precast Concrete Components below. Clear cover placement tolerances shall be in accordance with the SSBC.

These values shall be the basis for design and shall be specified on the Detailed Design Drawings unless noted otherwise on Department standard drawings.

Clear concrete cover for precast girders based on TEC or MOTT precast components may be as shown on the standard drawings and may be lower than the values specified in CHBDC. Using these standard girders requires Approval.

Where more than one concrete cover reference appears applicable to a given situation, the greater clear concrete cover value shall be used. The clear cover for anchorages and mechanical connections shall be those specified for reinforcing steel.

Table 5-8: Clear Concrete Cover for Cast-in-Place Concrete Components

LOCATION	CLEAR CONCRETE COVER
Bottom layer of approach slab on clean granular fill and polyethylene sheeting.	50 mm

Table 5-9: Clear Concrete Cover for Precast Concrete Components

LOCATION	CLEAR CONCRETE COVER
Precast concrete partial depth deck panels – measured to pretensioning steel.	40 mm
MSE wall precast concrete fascia panels – measured to reinforcing steel.	50 mm

5.10 Bearings and Joints

All steel components of bearings and joints – whether exposed or embedded - shall be protected against corrosion. The corrosion protection system shall be one of the following:

- Hot-dip galvanizing in accordance with ASTM A123.
- Metalizing to AWS C2.23 with a minimum zinc coating thickness of 0.2 mm.
- A coating system which is selected from the MOTT Recognized Product List or the TEC Products List.

While sealed deck joints are intended to be waterproof, they often leak and require maintenance over time. In response, components below deck joints shall be designed and detailed considering the potential for water ingress from the joint above. Design of joints shall also accommodate access for rehabilitation or replacement of components that may require replacement over the life of the bridge.

6. SUBSTRUCTURES, FOUNDATIONS AND EMBANKMENTS

6.1 General Requirements

Bridge substructure components provide support to the superstructure. Substructure elements experience differential and global settlement, rotation or tilt, and other stressors. When these become excessive, they may cause premature deterioration of substructure and superstructure components which can result in costly repairs or reduced bridge service life. The Department has experienced the following problems with substructures:

- Abutment or pier settlement, due to either inadequate fill compaction or underlying permafrost degradation.
- Scour exposing pier foundations, and in extreme cases scour or erosion at abutment locations, including above riprap level, due to aufeis accumulation.
- Progressive headslope movement or approach fill failure due to geotechnical issues that push and tilt substructure elements.
- Bridge or highway drainage around abutments that undermine approach slabs, abutment seats, drain troughs, and general slope erosion.
- Translations or rotations of earth retaining structures at abutments or piers.
- Pile foundation construction problems, such as inadequate driven pile embedment into permafrost that was not identified by site investigation.

New designs shall fully address the above-mentioned issues to avoid their occurrence.

Bridge substructures, foundations and embankments shall be designed in accordance with the CHBDC and supplemented as required by the Canadian Foundation Engineering Manual. In accordance with CHBDC Clause 6.8 (Design liaison, contract documentation, and support during construction), it is expected that the Geotechnical Consultant be fully involved during the detailed bridge design, including review of the final embankment and foundation design drawings, and during bridge construction (particularly pile installation). Where scour and erosion issues are a concern, it is expected that a hydrotechnical engineer will be fully involved during the conceptual and detailed bridge design stages.

Further to Clause 6.5.1 (Consequence classification) of the CHBDC, all bridge foundations, embankments and earth retaining structures shall be classified as having a “typical consequence” level unless otherwise Approved.

Geotechnical investigations shall be carried out for all structures, in accordance with the project-specific geotechnical terms of reference provided by the Department. For sites with permafrost, CAN/BNQ 2501-500/2017 (Geotechnical Site Investigations for Building Foundations in Permafrost Zones) shall also be followed. The selection of representative or “characteristic” geotechnical parameters used to determine foundation capacity shall be based on the results of appropriate field and laboratory investigations and shall represent the Geotechnical Consultant’s “best estimate” of the likely values of the parameters, considering all the factors that may influence the soil properties. The geotechnical report shall address all items identified in the CHBDC Clause 6.7 (Geotechnical report), and Clause 6.13.2 (Geotechnical investigation) where fully or semi-integral abutments may be considered for the bridge.

The key geotechnical parameters shall be presented in a table on the Detailed Design Drawings. This includes test hole information as well as geotechnical foundation design parameters (e.g., skin friction, end bearing, modulus of subgrade reaction, down drag, group effects, estimated settlements). The Canadian Foundation Engineering Manual (CGS, 2023) provides basic guidance on the development of geotechnical design parameters for permafrost conditions.

Seismic hazard evaluations are to be performed as per CHBDC Clause 4.4.3 (Seismic hazard). Site class, fundamental period of the structure, design spectral acceleration, and seismic performance category shall be specified in the general notes section of the foundation drawing.

Deep foundations (piles) shall be used for all bridge foundations unless otherwise Approved. Spread footings will be considered for Approval where a pile foundation is found to be not feasible or not economic, provided that the risks due to scour, slope stability and other factors can be controlled to acceptable levels. Typically, spread footings are only economical if the competent bearing surface is close to the ground surface. Construction risks related to the size and depth of excavations, and potential for ground softening due to water exposure during construction need to be fully considered by the Consultant before a spread footing is proposed.

For additional guidance on setting the depth of in-stream foundation elements to minimize the risk of scour or on designing rock protection for stream related substructure elements, refer to the TEC Bridge Conceptual Design Guidelines.

6.2 Piling

The Department's preferred pile type is driven steel piles; other pile types shall be used only if Approved. Other pile types are not addressed in detail in these guidelines. Before applying for Approval, Consultants contemplating the use of other pile types shall consider technical feasibility, practical constructability, and economics as they relate to the expected site conditions.

The design pile tip elevations shall terminate a minimum of 3.0 m above the available geotechnical test hole bottom elevation. This requirement is intended to reduce risk and uncertainty in the bidding process and to reduce the occurrence of claims based on differing soil conditions. If the design requires a lower design pile tip elevation, additional geotechnical test hole information must be obtained to confirm the geotechnical parameters used for the pile design.

The following design pile load information for both abutments and all pier piles shall be shown in the 'General Notes' on the appropriate 'Information Sheet' Detailed Design Drawing:

- SLS permanent loads only
- SLS extreme loads
- ULS permanent loads only
- ULS extreme loads (Combination ULS#)

PDA testing and some other pile driving parameters need to be specified in the Special Provisions; refer to the SSBC for more information.

6.2.1 Driven Steel Piles

Driven steel pipe piles and driven steel H-piles are considered acceptable pile types. Driven steel H-pile standard sizes should be as shown on standard drawing *S-003 (Steel Pile Details)*. Designs incorporating other pile types or sizes require Approval.

Driven piles may be considered in "warm" permafrost conditions, where the high unfrozen moisture content of the soil allows it to be somewhat pliable (i.e., not "hard" frozen). Pipe piles are preferred in this situation, particularly if a pilot hole is required to facilitate installation; the use of pilot holes with H-piles results in a significant void around the pile that has to be filled, whereas a pipe pile permits a better match between pilot hole and pipe sizes.

Welded pile splices are not desirable where tensile or flexural capacity is critical to the integrity of the structure (e.g., within the flexing length of integral abutment piles). However, as a result of the variability of pile driving, it is difficult to completely avoid splices in tension zones. As a result, the SSBC allows pile splices to occur in tension zones but requires 100% of these splices to be tested to confirm the quality of the splice. The Detailed Design Drawings must identify the extent of the tension zones (direct or flexural) and include a note stating that all welded splices within the tension zone shall require testing. The following note is an example:

> "ALL PILE SPLICE WELDS THAT ARE REQUIRED WITHIN THE TOP "X" METERS OF THE PILE ARE TENSION SPLICE WELDS AND SHALL BE TESTED TO CONFIRM THE QUALITY OF THE SPLICE"

6.2.2 Pile Capacity

In accordance with SSBC Section 3 (Foundation piles), a WEAP analysis is the Department's preferred approach for determining the geotechnical capacity of driven piles. This approach requires close coordination between the bridge engineer and the geotechnical engineer during design and construction. The 'Bearing Formula' shall only be used for small bridges if Approved and requires that the Consultant develop a site-specific Special Provision indicating that the 'Bearing Formula" is to be used.

Depending on the cost of dynamic and static load tests, the economics of pile design may be improved by increasing the relevant ultimate geotechnical resistance factor in accordance with CHBDC Table 6.2. Consultants shall consider the cost of carrying out a more detailed geotechnical investigation and/or test in order to reduce the pile costs. This should be considered for all larger river bridges. A discussion of the rationale used to balance the quality and quantity of geotechnical information collected against the potential foundation costs shall be provided to the Department.

Pile capacity may be determined as described in Sections 6.2.2.1 to 6.2.2.3.

6.2.2.1 Static Analysis

A static analysis uses empirical equations and correlations to estimate the pile capacity. This is typically used where the pile capacity is determined through correlation with the Standard Penetration Test (SPT). The pile design may be based on a geotechnical resistance factor using a 'typical' degree of understanding in Table 6.2 of the CHBDC. In order to utilize the 'high' degree of understanding, geotechnical parameters should be based on the Cone Penetration Test (CPT).

If, for whatever reason, it is not possible to obtain geotechnical boreholes at each foundation element, the pile design at locations where boreholes are not available shall be based on a geotechnical resistance factor using the 'Low' degree of understanding.

6.2.2.2 Dynamic Analysis

Dynamic analysis is typically used for driven piles using energy equations to estimate the pile capacity. A WEAP analysis is the Department's preferred approach for dynamic analysis.

The following options exist for selecting the appropriate geotechnical resistance factor when a WEAP analysis is used:

- If the hammer efficiency is known (i.e., will be measured/confirmed during driving), the pile design may be based on a geotechnical resistance factor using the 'High' degree of understanding in Table 6.2 of the CHBDC.
- If the hammer efficiency is not measured during driving, the pile design shall be based on a geotechnical resistance factor using the 'typical' degree of understanding in Table 6.2 of the CHBDC.

6.2.2.3 Pile Driving Analysis (PDA Testing)

Dynamic tests are typically used for driven piles to obtain a greater understanding of the pile capacity.

If a minimum of 10% of the piles at each substructure element are tested (minimum 1 per location), the pile design may be based on a geotechnical resistance factor using the 'typical' degree of understanding from Table 6.2 of the CHBDC.

If a minimum of 20% of the piles at each substructure element are tested (minimum 2 per location), the pile design may be based on a geotechnical resistance factor using the 'High' degree of understanding from Table 6.2 of the CHBDC.

Dynamic test results on piles installed in permafrost should be reviewed and interpreted by a geotechnical engineer experienced with permafrost applications.

6.2.3 Drilled Cast-in-Place Concrete Piles

While not covered in detail by the guideline, drilled cast-in-place concrete piles are a common foundation alternative that may be Approved in some cases.

Notwithstanding Clause 8.14.2.1.1 of the CHBDC, the clear spacing between parallel bars or parallel bundles of bars, including splice zones, in drilled cast-in-place concrete piles, shall not be less than:

- 5 times the maximum size of the coarse aggregate; and
- 125 mm.

In order to facilitate the concrete flow, consideration should be given to avoid using multiple layers of reinforcing bars in drilled shaft piles and should be replaced wherever possible by using higher strength bars, bundled bars, and/or increasing the diameter of drilled shafts.

The Consultant shall refer to FHWA document FHWA-NHI 18-024 (Drilled Shafts: Construction Procedures and Design Methods) for additional recommendations.

6.3 Bridge Piers

The tops of pier caps shall have a wash slope of 3% from the middle of the cap to the outside of the cap.

The upstream face of river piers shall be protected by an embedded galvanized nose plate designed to withstand ice forces. Nose plates should typically extend from a minimum of 600 mm below grade to at least 600 mm above the design surface water elevation. However, special studies are required at locations with the potential for ice jam formations, ice build-up, or other situations that may lead to ice build-up above the design surface water elevation.

For monolithic pier diaphragms which are cast around girder ends, the girders shall be erected on a minimum 150 mm high plinth to provide sufficient clear space between the girder underside and previously cast supporting concrete, to facilitate flow and consolidation of concrete under the ends of the girders.

For pier components, where the minimum dimension of the concrete component is greater than or equal to 2.0 m, the Consultant shall specify mass concrete for the component, in accordance with SSBC Section 4 (Cast-in-place concrete).

6.4 Bridge Embankments/Abutments

6.4.1 General

The Department permits conventional abutments and integral abutments (both semi-integral and fully integral).

Conventional abutments are those where superstructure movements and rotations are isolated from the abutment through the use of bearings and expansion joints. Semi-integral and fully integral abutments are discussed in Section 6.4.3.6 (Integral Abutments).

6.4.2 Embankments

Embankments shall include bridge headslopes, approach fills within 10 m from the end of the bridge, transitions between the approach fill sideslopes to the headslopes, and any earth retaining structures within this area.

The design of the bridge embankments shall account for stability, long-term settlements, and wall deformations. Stability analyses shall be carried out to determine that the embankments have acceptable short-term and long-term stability. The global stability of bridge embankments shall be designed for a minimum factor of safety against failure of 1.5.

Deformations of the embankment (including settlement and lateral movements) shall be determined using appropriate deformation analyses, with representative soil parameters derived from site-specific geotechnical investigations and local experience. The expected range of embankment displacements including settlement and lateral movements shall be considered in the design of the bridge and shall provide for acceptable structural and aesthetic performance of the embankments as well as the bridge. Embankments at bridge abutments shall be designed such that approach slabs, deck joints, bearings, barriers, and integral abutment piles in casings will operate as intended by design without imposing excessive stresses on the structure or requiring premature replacement of any bridge superstructure or substructure components due to long-term movements of abutment seats. The structural design shall include soil structure interaction analysis where appropriate. Mitigating measures such as early fill placement, temporary surcharges, excavation and replacement of inadequate base material, wick drains, stone columns, lightweight fill, or soil reinforcement shall be carried out where necessary to limit long-term movements. In addition to the CHBDC requirements, differential settlement between the bridge structure and the approach fills shall not result in a deviation of more than 0.5% from the roadway design grade – i.e., the expected settlement at the end of the approach slab shall not exceed 0.5% of the approach slab length.

Conservative estimates of the long-term vertical, longitudinal and lateral movements of the headslopes and earth retaining structures that will follow after completion of construction shall be made. These movements shall be estimated at the elevations of deck joints, bearings, and tops of piles as applicable. Joints, bearings, and piles shall be designed to accommodate these long-term movements over and above cyclical thermal movements and permanent prestressed girder creep shortening, in addition to an allowance for construction tolerances. The long-term movements incorporated into the design shall be identified on the Detailed Design Drawings.

Silt material specified as "ML" or "MH" material (in accordance with the "Modified Unified Soil Classification System") should not be used in the construction of the bridge embankment.

Cellular concrete and expanded polystyrene (EPS) blocks may be considered for lightweight abutment fill but would require Approval. Proposals for the use of these materials must be submitted to the Department with well thought out details to ensure long-term performance.

Important considerations would include:

- Protection of EPS blocks from exposure to hydrocarbons.
- Erosion due to leakage of highway drainage around abutments and wing walls.
- Effects of differential settlement and other movements.
- The buoyancy of EPS blocks when submerged.
- Compatibility of material properties (such as long-term creep and short-term elastic characteristics).

The geometry of embankments and earth retaining structures shall be sloped so that all drainage is directed away from bridge abutments. The drainage path shall be protected appropriately to prevent erosion.

6.4.2.1 Headslopes

The Department's preference is to specify open headslopes wherever practical.

Headslopes shall be protected with appropriately sized riprap protection for watercourse crossings in accordance with Section 4.1.3.2 (Scour and Erosion Evaluation). The possibility of aufeis accumulation and early spring scour should be considered.

Headslopes shall not be steeper than 2H:1V, unless Approved.

6.4.2.2 Inspection Access

Headslopes and earth retaining structures at abutments shall be configured to facilitate inspection of the girder ends, bearings, expansion joints, and abutment seats. This inspection access shall not require the use of ladders or any specialized equipment, such as snooper trucks or fall protection.

For bridges with open headslopes, include the following:

- Provide a minimum 600 mm wide horizontal bench in front of abutment seats.
- Limit the abutment seat height to the range of 600 mm to 1800 mm above the bench elevation.

For bridges with abutments behind independent earth retaining structures, inspection access shall include the following:

- Provide a horizontal bench suitable for inspection access not less than 600 mm wide in front of the abutment seat.
- Limit the abutment seat height to the range of 600 mm to 1800 mm above the bench elevation.
- Provide a continuous safety railing at the outside edge of the bench, designed as a “guard” in accordance with Part 9 of the current NBCC, having a minimum height of 1070 mm.

Further to the above, for side-by-side girder or solid slab bridges used in combination with integral or semi-integral abutments, short abutment seats can be problematic for inspectors. For these bridges, the height from the top of the grade or the top of the walkway to the underside of girders/slab shall be nominally 1500 mm.

6.4.3 Abutments

Any bridge components located immediately behind earth retaining structures, such as abutment seats, integral cantilevering wing walls, abutment deck joints, abutment bearings and traffic barriers, shall be designed to accommodate any movements resulting from vertical or lateral wall displacements.

Access for inspection and maintenance shall be provided to bearings, expansion joints, and the abutment backwall.

If MSE and GRS walls are used behind abutments, the soil reinforcing straps shall not be attached to abutment foundations, seats, backwalls or wing walls to reduce lateral earth pressures (there is concern with excessive strains on the connections caused by long-term differential settlement, which would occur under most circumstances).

Bridge plaques and benchmark tablets shall be provided at bridge abutments in accordance with standard drawing *S-001 (Standard Asset ID Plaque And Bench Mark Tablet)* and *S-002 (Standard Culvert Asset ID Tag)*.

6.4.3.1 Abutment Seats

The tops of abutment seats shall have a wash slope of 3% towards the front of the seat.

Abutment seats shall be embedded by a minimum of 500mm below the top elevation of the required inspection access bench described in Section 6.4.2.2 (Inspection Access).

Where the minimum dimension of the abutment seat is greater than or equal to 2.0 m, the Consultant shall specify mass concrete for the seat, in accordance with SSBC Section 4 (Cast-In-Place Concrete).

6.4.3.2 Wing walls

Bridge ends shall have cast-in-place wing walls oriented parallel to the over-passing roadway.

For abutments without roof slabs, wing walls shall be cantilevered from the abutment seat or the superstructure end diaphragm.

For conventional abutments, when wing walls exceed 10 m in length, consideration should be given to designing the abutment with roof slabs and grade beams. See Section 6.4.3.3 (Roof Slabs and Grade Beams).

Wing walls shall extend beyond the top of the fill line by a minimum of 600 mm and shall be embedded a minimum of 600 mm below the top of the grade at all locations.

For integral abutments, wing walls shall meet the additional requirements of Section 6.4.3.6 (Integral Abutments).

6.4.3.3 Roof Slabs and Grade Beams

Abutments with roof slabs and grade beams are connected by the wing walls to form a rigid box. They are considered to be very stable and provide significant resistance to movements and rotations.

Access to the cavity below the roof slab shall be provided through the abutment backwall, with access positioned between girders and accessible through the abutment diaphragm.

Bridge abutments with roof slabs typically have an interim construction stage where the roof slab has been cast, but a void at the end of the slab (the deck joint block-out) remains for quite some time before the deck joint is cast into place. The roof slab with the deck joint block-out must be designed and detailed to ensure that the remaining slab can support all necessary construction loads.

6.4.3.4 Approach Slabs

Approach slabs shall be incorporated into all new bridge designs unless otherwise Approved.

Approach slabs shall have sufficient length to limit their rotation due to settlement to 0.5% and shall have a minimum length of 6000 mm (measured parallel to the centreline of the roadway). Approach slabs for integral abutments shall be extended at least 1000 mm past the end of wing walls.

Where approach fills have been placed in the winter, approach slabs should not be constructed until the second summer after fill placement (where the project schedule permits).

Approach slabs shall not be constructed with integral barriers or curbs unless Approved.

Approach slab thickness shall be as determined by the Consultant but shall be a minimum of 300 mm.

Approach slab reinforcement shall be as determined by the Consultant, but the bottom steel shall not be less than 20M @ 150 mm placed parallel to centreline of roadway and 15M @ 150 mm placed parallel to abutment backwall, and top steel shall not be less than 15M @ 300 mm each way.

Approach slabs shall be connected to the bridge in a manner that provides for free hinging rotation without causing restraining moments and forces.

Bridge approach fill may sometimes settle more than the specified design value based on 0.5% rotation of the approach slab. In this case, pavement smoothness is expected to be restored either by mud jacking of the approach slab or by milling and re-paving of the ACP wearing surface. Mud jacking is not recommended for integral abutments due to the concern of bonding to the underside of the slab and preventing thermal movement. Settlement on gravel roads can be adjusted by adding additional gravel surfacing material and re-grading of the approaches to tie into the bridge deck elevation. Chipseal pavement would need to be removed, the base layer re-graded and chipsealed to restore surface smoothness.

6.4.3.5 Abutment Drainage Details

Abutment drainage details shall be incorporated into the design of abutments and shall include the following:

- The joints around the approach slab shall be well-sealed to prevent water infiltration.
- A secondary drainage system consisting of granular backfill, sheet wall drains, and sub-soil weeping drains shall be provided to collect, channel, and remove any seepage. Sheet wall drains are typically spot glued to the earth face of the abutment seat and wing walls to intercept and channel seepage into a perforated weeping drainpipe complete with filter fabric sock. The pipe shall have a minimum positive drain slope of 2% and shall be daylighted onto the headslope or side slope, and a galvanized screen shall be installed on the pipe opening to prevent small animals from entering the pipe.
- Sheet wall drains shall be omitted for MSE or GRS wall abutments with steel soil reinforcement, to eliminate a direct path for leakage to the steel reinforcement. This means that any leakage will be forced to filter through the granular backfill. The steel reinforcement shall also be protected by an impermeable membrane.
- Clean, well-graded, crushed granular backfill with a maximum aggregate size of 25 mm (Des 2, Class 25) shall be provided behind the abutment seats and wing walls complete with perforated weeping drains under the abutment seat and wing walls.
- For abutments with earth retaining structures, drainage from deck joints and deck wick drains shall be drained into downspouts that extend down the front face of the wall and discharge away from the wall. Drainage swales along the top of wing walls shall always drain laterally away from the roadway and bridge abutments. Refer to the requirements in Section 8.3 (Earth Walls: MSE and GRS).

Bridge deck drainage shall always be controlled and channeled away from bridge components. Refer to Section 14 (Bridge Drainage).

6.4.3.6 Integral Abutments

Integral abutments shall meet the general requirements for abutments identified in Section 6.4 (Bridge Embankments/Abutments) and shall be designed in accordance with CHBDC Clause 6.13 (Integral and semi-integral abutments).

Integral abutments (both fully and semi-integral) shall be used whenever practical in order to take advantage of the benefits listed in the following:

- Reduced initial costs and long-term maintenance costs.
- Locates the control joints away from the girders and other critical structural components.
- Elimination of deck joints, resulting in fewer tolerance restrictions and faster construction.
- Smoother ride across bridges due to continuous wearing surface.
- Reduced number of foundation piles.
- Abutment ends are buttressed between headslope fills at each end, resulting in less potential for superstructure shifting and tilting of piers or abutments.

Fully integral abutments provide the following additional benefits:

- Improved resistance to uplift at abutment end.
- Reduced end span to interior span ratio may allow longer interior spans.
- Increased reserve load capacity and better load distribution, resulting in more resistance to damaging effects of unexpected overloads.

Integral abutments shall not be used when MSE or GRS walls are incorporated into the bridge abutment as wing walls. In these situations, it is too difficult to control water runoff at the sides and ends of the approach slabs, creating the potential for erosion and deterioration issues. Integral abutments should be avoided in situations where excessive settlement of the approach roadway and/or fills is expected.

Integral abutments shall meet the following requirements:

- Integral abutments should not be used for bridge lengths greater than specified in Table 6-1: Maximum Length for Integral Abutment Bridges unless Approved. The difference in concrete and steel bridge lengths reflects the effect of the greater thermal mass of concrete and the greater sensitivity of steel in reacting to ambient temperature changes. The longer span limits in the second row of Table 6-1 require the use of a sleeper slab at the back of the approach slabs; refer to TEC standard drawing *S-1840 (Standard Waterproofing System for Deck and Abutments – Sheet 3)* for additional guidance.

  The bridge lengths shown are based on structures with equal stiffness in all supports leading to equal thermal movements at each abutment. For unsymmetric structures, the “thermal span” shall be taken as the distance measured from the superstructure location which experiences zero longitudinal movement under temperature changes to the furthest abutment bearings; this thermal span shall be limited to 50% of the lengths shown in Table 6-1.

Table 6-1: Maximum Length for Integral Abutment Bridges

Sleeper Slab Required at Approach Slab	Maximum Length For Steel Girder Bridges	Maximum Length For Concrete Girder Bridges
No	30 m	40 m
Yes	60 m	80 m

- Care must be taken for bridges with skew, the effects of skew, including the effects of earth pressure, the potential for twisting of the superstructure (in plan), and biaxial bending of the piles shall be analyzed and accounted for in all designs.
- Integral abutments shall not be used for bridges with skew greater than 15 degrees without Approval.
- The Geotechnical Consultant shall provide geotechnical information in accordance with CHBDC Clause 6.13.2 (Geotechnical investigation) and shall provide an assessment of headslope suitability for the use of integral abutments. The impact of headslope instability shall factor into the abutment type selection.
- The height of integral abutments and the length of wing walls should both be minimized in order to reduce the associated soil pressures and resistance to movement. As girders and abutment seats get deeper, the earth pressure resisting thermal expansion becomes considerably greater and harder to precisely quantify. For deeper girders, semi-integral abutments with partial depth backwalls may be used to minimize the earth pressure. In this case a conventional backwall is constructed but instead of extending to deck level, it stops at the underside of the approach slab and the approach slab is supported by the girders and spans across the top of the backwall.
- The height of the abutment seat above grade shall not exceed 1500 mm.
- Wing walls for integral bridges shall be turned back and oriented parallel to the roadway. Wing walls shall be attached to and cantilevered off the back of the superstructure end diaphragm. Maximum wing wall length shall be 8 m as measured from the back of the end diaphragm to the end of the wing wall.
- Integral approach slabs shall not be designed to move longitudinally in and out between stationary and parallel non-integral wing walls.
- Approach slabs shall extend at least 1000 mm beyond the ends of the wing walls.
- Two layers of polyethylene sheet shall be provided under the approach slab to minimize frictional forces due to horizontal movement. The connection between the approach slab and the superstructure shall be designed to resist these forces.
- The approach rail transition shall be rigidly attached to the ends of bridge barriers, regardless of whether the barrier ends are stationary or moving.
- Where barriers are permitted to be constructed integral with approach slabs, the design shall be such that loss of barrier height due to settlement and overlay does not exceed 30 mm over the life of the structure. The joint between the barrier on the approach slab and the barrier beyond the approach slab shall be kept sealed.

- The installation of expansion foam material behind integral abutments for the purpose of relieving earth pressures is not permitted for the following reasons: The soft material can get compressed or punctured by backfill compaction during construction. Eventually, cyclical movements may cause progressive plastic compression and promote unequal movements at the two abutments, resulting in a net shift to one side of the whole structure.
- For monolithic abutment diaphragms that are cast around girder ends, the girders shall be erected on a minimum 150 mm high plinth to provide sufficient clear space between the girder bottom and previously cast concrete. This is to ensure proper flow of concrete under the ends of the girders.
- Additional deck reinforcement shall be provided for resisting negative bending moments near the ends of the superstructure due to the flexural continuity of the girder-to-pile connection and the related torsional restraint of the stiff abutment diaphragms, adjacent girders, and earth pressure.
- Construction sequence can have a significant impact on the success of bridges with integral abutments. The Consultant must fully understand the potential effects of construction sequencing, shall choose the design approach that is most appropriate for each specific bridge, and must determine if and how the contractor is required to follow any procedures or provide any temporary measures to accommodate these effects. These requirements shall be clearly and explicitly stated on the Detailed Design Drawings. The effects of construction sequencing include but may not be limited to the following:
    - Effects of deck pour causing girder end rotation.
    - Effects of thermal changes. Thermal cycles during construction result in girder end rotations and lateral and longitudinal movements of the girders. This can then give rise to differential movement between the girders and the abutment with the girders shifting or the abutment seat moving. There are several options for accommodating these effects, including rigidly connecting the girder ends to the abutment with anchor rods (for lateral and/or longitudinal movements), or by simply allowing the girder ends to float on temporary bearing pads with or without temporary construction restraints.
    - Effects of post-tensioning on the abutment. Consideration shall be given to completing any post-tensioning before making the permanent connection between the girder end and the abutment. Temporary sliding bearings and/or anchoring of the abutment seat may be required to accommodate longitudinal post-tensioning effects.
    - Stability of the structure during all stages of construction.
    - Backfilling behind abutments. Backfilling behind abutments should follow a balanced sequence so that the superstructure is not pushed out of position by backfilling operations.
- The designer must consider the construction timing when designing integral or semi-integral abutments to allow for adequate thermal movement ranges during the service life of the structure.
- To achieve a successful and durable design, good drainage details must be incorporated. The goal is to prevent surface drainage from going below the surface at the abutments.
- The joints between the approach slab and the wing walls should be sealed to prevent water infiltration. Even the best attempt to seal joints cannot prevent all leakage. It is therefore important to design a secondary system of sub-soil weeping drains to collect, channel and remove the seepage. Drainage at the control joints must be collected and directed to the side slopes.

- Surface drainage at the ends of wing walls is difficult to control because the wing walls move with the superstructure. This is further complicated on bridges with concrete approach barriers. Details for maintaining water tightness between the thermally cycling wing wall barrier and the stationary approach shall be carefully thought through.
- The joint gap at the back of approach slabs may open up more than desirable during the first few years after construction, due to creep and shrinkage shortening of prestressed girders. However, this is a one-time permanent irreversible opening. The opened gap can be repaired as follows for systems with ACP wearing surfaces and roadway approaches:
    - Remove a minimum 1200 mm wide strip of ACP over the length of the joint.
    - Clean out the existing joint gap down to the top of the existing transverse wick drain; Replace the transverse wick drain if damaged.
    - Sandblast the exposed vertical face of the sleeper beam upstand (if present).
    - Install and spot glue new 20 mm asphalt-impregnated fibre board (AIFB) to the end face of the approach slab.
    - Fill the remaining gap with an approved chemical grout such as Set 45.
    - Repave and saw cut the pavement over the top of the AIFB and reseal the joint with asphaltic hot pour rubberized crack sealant.

  The same procedure can be applied to locations with a concrete or gravel wearing surface; simply ignore requirements relating to ACP or wick drain.

The following additional requirements shall apply to fully integral abutments:

- Abutment piles must be designed for vertical dead and live loads as well as moments that result from the superstructure movement and rotation. For bridges with larger thermal movements, the moment capacity can often limit the pile design. The designer may choose the option of carrying out a more refined pile design analysis to design the pile with the development of a plastic hinge. When this approach is taken, care must be taken with this method as it requires a more comprehensive analysis and careful design of the pile to ensure that the pile can develop a fully plastic stress distribution without flange local buckling.
- Piles must be sufficiently flexible to allow the abutment to move without developing large forces in the superstructure. Hence, the abutment foundation shall consist of a single row of piles.
- When H-piles are used, they shall be oriented for weak axis bending. For skewed bridges, it is customary practice to orient the H-piles in the direction of the expected superstructure thermal movement. For small skew, it is typically acceptable to orient the H-piles parallel with the superstructure. For larger skew, a more comprehensive analysis is recommended to determine the additional forces that may develop. H-piles shall be embedded a minimum distance of two times the pile depth into the abutment seat.
- When pipe piles are used, they may require cast-in-place concrete fill to some depth of the piles to allow for formation of a plastic hinge near the top of the pile; this need should be considered early in the design process in case that concrete is not be readily available at the site.
- Since the girders are erected on abutments with a single line of flexible piles, special care must be taken to ensure abutment stability for all stages of construction. If temporary support is required, the Consultant shall state the requirements on the Detailed Design Drawings or in the Special Provisions.

- For structures requiring a sleeper slab below the approach slab, or when surrounding soils will restrict pile movement, piles shall be installed inside permanent steel casings, and the casings shall be filled with Styrofoam beads. Beads shall be new "Storopack" virgin polystyrene 14.4 kg/m^3 (0.9 pcf) filler beads, nominal diameter of 5 mm, or approved equivalent, only. Reground or recycled polystyrene material is not acceptable and shall not be used. In cases where water ingress into the casing could be detrimental to the performance of the pile, consideration shall be given to plugging the top, bottom, or both ends of the casing.
- Steel casings shall be designed to last the same life as the bridge, and an appropriate sacrificial corrosion thickness and/or galvanizing shall be provided.
- When fully integral abutments are used at streams, the Consultant must consider the elevation of the top of the pile casing in relation to the normal stream and groundwater elevations to ensure that water will not get trapped in the casing, which could affect the pile durability as well as the ability for the pile to freely flex when the water freezes.

7. BRIDGE-SIZED CULVERTS

7.1 General

Bridge-sized culverts are any culverts with a minimum span of 1.5 m or larger. The maximum permissible span of culverts shall be 6.0 m. Where culverts 3 m or greater are proposed, the feasibility and cost-benefit of a short-span bridge should be investigated as an alternative option. Larger culverts have proven to be challenging with local soil conditions and would require explicit Approval. General culvert design considerations, as well as design considerations for trenchless installation, are given in the following sections.

7.2 Culvert Design Considerations

Drawing *T-001 (Installation of CMP and SPCMP Structures)* provides guidance for the design and installation of culvert structures. This drawing is applicable to bridge-sized culverts with a diameter of 3 m or less, with site-specific details required for culverts with greater diameters and all open bottom culverts.

Design considerations for culverts include:

- Structural Plate Corrugated Metal Pipe (SPCMP) shall be used for all bridge-sized culverts. Corrugated Metal Pipe (CMP) may be used only for culverts smaller than 1.5 m in diameter. The SSBC includes steel and aluminum culverts under SPCMP and CMP culverts.
- Culvert inverts should be buried one quarter of the rise (D/4) below the average natural streambed, up to a maximum depth of 1 m, to allow for future streambed changes and ensure biological connectivity. Exceptions to the recommended burial depth may be considered when site-specific features require special attention such as shallow competent bedrock, environmental requirements, permafrost, or historical stream degradation.
- Settlement at culverts can cause sagging of the culverts resulting in ponding water and uplifted/perched culvert ends. Camber should be included in culvert design and based on site-specific geotechnical information, subgrade conditions, fill height, backfill material and the anticipated settlement.
- Culverts have fixed opening dimensions with transition zones at both ends of a culvert, as the channel shape moves from the bevel end to a trapezoidal shape that can fit the natural stream. Transition zones also connect the culvert invert elevations to the natural streambed elevations. In some cases, berms perpendicular to the roadway may be required to support the protection works. Channel realignments may also be beneficial in some cases, to reduce culvert skew/length, improve flow alignment, or reduce property impacts. Channel realignments at permafrost sites should be carefully considered to limit permafrost degradation by excavating into undisturbed ground.
- A slope of 4:1 is preferred for side slopes as this slope is "recoverable" and guardrails are not required, provided culvert ends are outside the clear zone. If a 4:1 slope is not possible (due to right-of-way constraints, for example) a 4:1 slope through the clear zone, followed by 3:1 outside the clear zone (with no guardrail) is often a cost-effective solution. For high fills, steeper side slopes should be evaluated in terms of cost, roadside safety, and geotechnical conditions; a steeper side slope with guardrail protection may be the preferred solution.
- Minimum soil cover for steel culverts shall be in accordance with the CHBDC or 1000 mm, whichever is greater. The minimum cover shall be taken as the least dimension between the crown of the culvert and the

pavement surface at the edge of the roadway shoulders. When barriers are proposed, the cover depth shall be sufficient to allow for proper foundation embedment of the barrier posts. Soil cover less than the minimum requires Approval.

- Multiple culverts shall only be used if they fit the natural channel (i.e., the bed width created should be similar to the natural channel). Flow expansion and contraction associated with structures being too small or too large in comparison to the natural channel should be avoided to minimize erosion and scour risks. In cases where multiple culverts are used, the use of fewer, but larger culverts should be considered to reduce low velocity zones between culverts. Culvert design should consider the natural sediment transport regime for a range of flows and consider the risk of aggradation and scour at inlets and outlets.
- Horizontal spacing between adjacent culverts shall be at least 1.0 m, or as recommended by the culvert manufacturer, to allow for the soil envelope to be established and properly constructed. Site-specific backfill design details are required for sites with multiple pipes.
- Open bottom structures shall not exceed 6 m in span but shall be larger than the natural channel bed width and shall have mitigation measures to prevent failure due to scour or erosion. Open bottom culverts shall not be used for stream gradients exceeding 2.5% to reduce the likelihood of scour undermining the foundations. Foundations for open bottom culverts may be spread (strip) footings or pile caps supported by piles. Scour and erosion risks must be carefully considered during the selection of the foundation type for open bottom culverts.
- The presence of organic or frost-susceptible natural soils (silt and clay) and uneven confining pressure can result in differential frost heave at culvert crossings, where the ground at the ends of the culvert heaves more than under the embankment. This can result in progressive jacking of culvert ends. This process can be exacerbated by hydraulic effects if groundwater flow is impeded at the embankment and forms massive ice deposits (injection ice) below grade. The potential for these processes to occur should be considered during design and mitigated.
- Guidance on available culvert types and materials can be found in Section 18 (CMP and SPCMP structures and trenchless installation of steel pipe) of the SSBC. The selection of the optimal culvert configuration that meets the required service life should be based on lifecycle analysis and site-specific constraints.
- Culvert upsizing to allow for future lining is recommended as a cost-effective strategy at sites where open-cut replacement would be expensive or require extensive traffic accommodation (high fills, high traffic volume) and should be discussed with the Department.
- Cathodic protection systems are not permitted as service life extension strategies, as they are difficult and costly to maintain/operate.
- The difference in thermal conductivity between different culvert materials is not considered to be a significant factor for heat flow in horizontal structures, so no constraints on the selection of culvert type at permafrost sites is necessary. Similarly, pipe coatings are not necessary for thermal insulation purposes.
- Headwalls and wing walls should be provided to improve culvert entrance hydraulics and to protect road embankments from direct impingement of flowing water. Headwalls can also be employed to resist ice jacking effects and protect culvert ends from impact deformation.

Further guidance regarding bridge-sized culverts and buried structures can be found in the TEC Design Guidelines for Bridge Sized Culverts. Other sources for information include the Corrugated Steel Pipe Institute, the Canadian Concrete Pipe and Precast Association, and local suppliers.

Any items that require shop drawings in addition to the requirements included in Section 18 of the SSBC shall be specified in the Special Provisions.

For culverts conveying water, management of the water is often a governing consideration. The Consultant shall specify parameters for water management during construction in the Special Provisions.

7.3 Culvert Design and Construction Considerations in Northern Regions

This section discusses some practices that can be considered non-routine, or that differ from what might be considered good practice in temperate regions.

At sites with permafrost, it is advisable to align the culvert within the existing drainage path, as it is likely that the underlying permafrost has already been thermally impacted by the water flow, and less disturbance to permafrost may occur from the excavation activities associated with culvert installation. Similarly, when replacing a culvert, it is preferable to replace the culvert on the same alignment as the former, rather than shifting the culvert to facilitate construction. Constructing a new or replacement culvert along a new alignment, for the sake of construction expediency, at a site underlain by permafrost, can be expected to result in permafrost degradation and ground settlement following construction.

Sub-excavation of ice-rich permafrost below culverts should generally be avoided, as the backfill is likely to be more disruptive to permafrost stability than the original ground. If site investigation determines that the ice-rich zone extends to a defined, limited depth, then sub-excavation may be effective.

Winter construction is sometimes unavoidable for culvert installation and often required to protect the underlying permafrost. This means that backfill materials will be frozen at the time of placement and compaction. The backfill materials should be relatively “dry”, such that they do not contain excess ice. Use of ice-rich backfill is not acceptable.

Clean, open-graded aggregates have been used effectively for backfilling of culvert excavations in the winter, because they are not moisture sensitive and can therefore achieve a higher degree of compaction without moisture conditioning. However, such backfill materials are pervious, so should not be used for culvert bedding at sites underlain by permafrost. Well-graded, less pervious gravel should be used for culvert bedding at sites underlain by permafrost.

Clay plugs are not widely used in the north, primarily due to a general lack of sources of clay. Also, they are problematic to install, particularly in winter. A preferable solution is to accept that some water will bypass the culvert and use well-graded, relatively impervious gravel for culvert bedding. If there is sub-excavation beyond the normal bedding depth below a culvert, particular attention should be given to controlling groundwater seepage through this zone.

7.4 Trenchless Culverts

Trenchless installation methods provide a viable alternative to traditional open trench installation methods. Compared to open trench methods, the utilization of trenchless methods may have the following benefits:

- Significant reduction in impact to traffic.
- Avoid building a detour road/bridge.

- Minimize damage to existing infrastructure.
- Reduce grading work.
- Reduce environmental impact.
- Increase the safety of workers and the travelling public.

Additionally, there may be benefits to cost and schedule in some circumstances – particularly for deep culvert installations where open trench methods are not feasible. This discussion pertains to the installation of thick-walled welded steel pipe (WSP) by trenchless methods that may include auger boring, pipe ramming, pipe jacking, or pilot tube guided boring for casing diameters less than 3000 mm.

7.4.1 Applicability of Trenchless Installation

There is generally more risk associated with trenchless installation methods compared to conventional cut-and-cover techniques for highway culverts. Some of these risks may include differing ground conditions, surface heave or settlement, unknown obstructions along the proposed alignment, or existing or unknown buried utilities and infrastructure. While these risks are also common on cut-and-cover installations, they can generally be accommodated with minor cost and schedule implications as access is readily available. For a trenchless installation, these risks could require an intervention excavation, restarting the installation on a different alignment, abandonment of an installation, or requiring an unplanned and unscheduled detour – all of which could have significant financial and schedule implications. For these reasons, the Consultant should carefully consider when to specify trenchless installation methods and understand the risks associated with trenchless installations.

The following considerations may necessitate the use of trenchless installation methods:

- **Open Excavation is Not Feasible:** Crossing is located beneath existing utilities or other infrastructure, insufficient space to stage construction, overhead obstacles, or where excavation is not possible.
- **Detour is Not Feasible:** Closing the road is not possible or impractical due to the volume of traffic, space limitations, or rerouting of traffic not feasible, or where road closure results in a dead end.
- **Depth Exceeds Reasonable Excavation Limits:** Trenchless installations can become more cost-effective on deep fills where large open cuts lead to higher costs due to detour requirements, space implications, right-of-way constraints and equipment limitations.
- **Right of Way Constraints:** The typical highway right of way is 60 m (30 m either side of the centreline). For culverts under large fills the space required to facilitate the working room for trenchless installations will likely be outside of the right of way as the highway footprint is already very large. The trade-offs between temporary detours and temporary construction access outside of the right of way for trenchless installation methods should be evaluated to assist in deciding the preferred installation method.
- **Condition of the Existing Travel Surface:** The travel surface is new or recently paved and excavation or cutting the surface would result in potentially degrading the surface and impact the life expectancy of the road surface.
- **Social and Economic Impacts:** Where duration and extent of open trenching could have an adverse impact on adjacent residents, commercial establishments, or other stakeholders.
- **Environmental Impact:** Where the potential impact of the project and excavation operations could negatively affect the environment. It should be noted that even trenchless installations likely require some degree of instream work for culvert installation and setup areas.

It should be noted that trenchless installations will often cost more than open trench methods. The Consultant should undertake a detailed cost analysis of options, consider the potential risks and availability of contractors, and assess the feasibility of the installation before specifying trenchless installation methods.

7.4.2 Data Collection and Site Investigation

Data collection and site investigation are critical steps in the design of trenchless crossings where complexity can significantly vary between any two projects, and even between crossings within the same project. Proper judgement by the Consultant is essential to determine the level of complexity of a crossing, and to manage the risks and challenges for the site-specific crossing conditions through an appropriate level of predesign investigation.

Desktop Review: The Consultant should complete a desktop review of the crossing site to develop a baseline understanding of the general site conditions including existing underground and overhead utilities parallel and perpendicular to the considered crossing, approximate site topography, general subsurface conditions, site access, potential construction staging or working areas, and right-of-way requirements.

Site Reconnaissance: Following the completion of the desktop review, or as a supplement to the desktop review, the Consultant should conduct a site visit to the crossing site to develop a firsthand understanding of the site conditions and evaluate the potential site-specific hazards, challenges, and constraints.

Topographic Survey: A site survey should be completed to confirm the existing topography of the crossing site, and to confirm the locations of surface visible utility features and other existing features (trees, watercourses, etc.).

Hydrotechnical Assessment: Refer to Section 4.1 Hydrotechnical.

Geotechnical Investigation: Following the completion of the desktop review, a site-specific geotechnical investigation should be completed to confirm the ground and groundwater conditions, and the possible impacts of trenchless methods within the identified conditions. For highway crossings, test holes are generally drilled off the roadway, along the shoulders or ditches, to minimize impact on traffic. The number and terminal depths of test holes should be determined by the Consultant based on their understanding of the general site conditions, anticipated ground conditions, and crossing complexity. At a minimum, one test hole on either side of the crossing drilled to a terminal depth of 4.5 m below the anticipated base elevation of the working pits is recommended. Test holes should be off alignment to avoid potential pathways for groundwater seepage to the excavation face or inadvertent returns in the case of systems that utilize pressurized drilling fluid and/or lubrication. Test holes should be surveyed upon completion to record position and surface elevation and test hole logs should characterize the encountered strata by depth below the surface as well as elevation. Disturbed sampling should be completed to classify the encountered soils. Classification should be completed to the Unified Soil Classification System, recording the description of the ground response to trenchless excavation in terms of the Tunnelman's Ground Classification System (TGCS) (Terzhagi 1950 and Heuer 1974) may also be beneficial. Undisturbed sampling is required if laboratory strength testing is requested by the Consultant. Field tests, such as in situ vane shear testing and pocket penetrometer testing, may also be completed to provide an indication of shear strength and unconfined compressive strength. Standard penetration testing (SPT) should be completed to provide an indication of relative density in cohesionless soils and consistency in cohesive soils. Test pits may also be beneficial for strata suspected to contain cobbles and boulders. Laboratory and field testing to determine soil properties should be completed in accordance with the latest ASTM International standards. Relevant soil properties to assess trenchless feasibility and complete trenchless design are summarized as follows:

- Cohesionless Soils: particle-size distribution (gradation), unit weight, hydraulic conductivity, shear strength parameters (cohesion and soil friction angle), and density (generally in terms of SPT blow counts/N-values).
- Cohesive Soils: moisture content, Atterberg limits, unit weight, shear strength parameters (undrained shear strength), and consistency (generally in terms of SPT blow counts/N-values or unconfined compressive strength).
- Bedrock: lithology, rock quality designation (RQD), weathering index, fracture index, unit weight, uniaxial compressive strength; and CERCHAR abrasivity index (ASTM D7625).

7.4.3 Design Considerations

Trenchless crossing design should be completed by experienced design professionals with an understanding of the site-specific constraints, ground and groundwater conditions, and the overall crossing complexity and risk.

Existing Embankment: If available, as-built drawings and construction documentation should be reviewed by the Consultant to understand the existing embankment materials. These records may provide insight into the source of the embankment material (i.e. rock quarry, borrow pit, or right-of-way excavation) and potential risks or challenges during installation (i.e. boulders/rock in the fill). The Consultant should discuss the project with local maintenance staff and the Department representatives early in the project as they can provide valuable anecdotal information and local, site-specific knowledge that may not be captured in the as-built and construction documentation.

Horizontal Alignment: The horizontal alignment of the crossing should be designed by the Consultant with consideration of existing rights-of-way, existing utilities and culverts, available space for working pits, and highway width. Crossings should be designed as close as possible to 90 degrees (perpendicular to the highway) to minimize crossing length and the associated potential area of disturbance. Crossing drive lengths should be limited to the reasonable capabilities of the anticipated trenchless installation methods. Sufficient horizontal separation between parallel crossings should be maintained to prevent failure by soil slip between the crossings.

Vertical Alignment: The vertical alignment of the crossing should be designed by the Consultant with consideration of the anticipated ground and groundwater conditions, estimated magnitude of ground deformation, boundary conditions (e.g., upstream, and downstream channel elevations), minimum slope for hydraulics, and slope limitations based on equipment capability. The vertical alignment should be designed within crossing horizon soils (the elevation band +/- one pipe diameter of the design vertical alignment) suitable for excavation by the anticipated trenchless installation methods; mixed face conditions should be avoided where possible. Where unavoidable, the Consultant should assess the potential impacts of the mixed face conditions, the anticipated excavation process, and any additional risk of ground deformation. The vertical alignment should be designed with continuous grade between working pits. The vertical alignment of the crossing should also be designed considering working pit depths, the ground and groundwater conditions, and areas of impact for ground deformation surrounding the working pits.

Pipe Size: Smooth wall steel pipes installed by trenchless installation methods should be designed by the Consultant with consideration of the required minimum hydraulic diameter, the anticipated ground conditions including the potential for cobbles and boulders or items sometimes buried in roadway fills, industry-standard nominal pipe sizes, and common pipe sizes for trenchless installations based on equipment capability. Where cobbles and boulders are anticipated, alternative alignments and crossing strata should be considered. If ground conditions with the potential for cobbles and boulders are unavoidable, a minimum pipe size of three times the largest obstruction should be considered to enable the removal of the obstruction from within the pipe, up to a maximum of 3000 mm.

Ground Deformation: Trenchless installations generate ground deformation both during and after installation. The magnitude of deformation can be reduced by the selection of the appropriate installation methodology for the ground conditions, good practices by the Contractor during installation, overcut size, and appropriately designed crossing depth. Settlement estimation should be completed by the Consultant or the project's Geotechnical Engineer, as part of the vertical alignment design. Allowable surface settlement limits should be determined in consultation with the Department. Surface settlement monitoring should be conducted on a project-specific basis as required in the project's contract documents. Monitoring should include surface points on- and off-alignment to capture the full surface trough width. Monitoring should be conducted at a predetermined frequency during the installation and for a period following the installation to capture both short- and long-term deformations. Baseline measurements should be captured prior to installation.

Working Space: Entry and exit working pits are required on either side of trenchless crossings. Working pit depth is based on the vertical alignment of the crossing and the topography on either side of the crossed feature. Working pit support of excavation design is the responsibility of the Contractor, however, the Consultant should assess the feasibility of constructing working pits for any given project as part of the project design phase. Considerations for working pits should include the ground and groundwater conditions, reasonable pit dimensions to accommodate smooth wall steel pipe segments and jacking/thrusting/ramming equipment, right-of-way space and easements, proximity to utilities and other site features, and setback requirements for working pits.

External Loads: Smooth wall steel pipes installed by trenchless installation methods should be designed by the Consultant with consideration of the reasonable long-term loads for the duration of the design/service life of the crossing (soil dead loads, traffic live loads, and surcharge loads) and the axial construction loads. Appropriate soil loading should be estimated by the Consultant based on the ground conditions and the design crossing depth; conservatively, prism load theory is recommended for design, however, load theories considering soil arching or soil strength may be considered when appropriate. Surcharge loads should be considered on a project-specific basis, based on the Consultant's understanding of the crossing site (e.g., surcharge pressure due to surface flooding).

Corrosion: Smooth wall steel pipes installed by trenchless installation methods are generally unlined and uncoated due to the risk of coating and lining damage during the installation. The pipe should be designed by the Consultant with an allowance for external and internal corrosion, through increased wall thickness, based on the design/service life of the crossing, the ground and groundwater conditions, and the contents of the installed pipe.

Pipe Design: Pipe wall thickness and specified minimum yield strength should be designed based on the long-term external transverse loading conditions determined by the Consultant. Design checks for performance limits should include ring deformation, wall buckling, and wall crushing. Performance limits that account for sidewall support from the surrounding soils should be determined using estimated soil stiffness values based on the density or consistency of the crossing horizon soils as determined in the crossing-specific geotechnical investigation. Pipe wall thickness and specified minimum yield strength should also be confirmed by the Contractor based on their equipment and installation methodology. Axial construction loads (jacking forces) should be estimated considering the pipe size, ground and groundwater conditions, anticipated installation methods, external and internal skin friction, as well as face resistance.

Design Standards: Design and trenchless installation of smooth wall pipes should be carried out in accordance with the most recent versions of the following relevant standards:

- ASCE. Horizontal Auger Boring Projects. ASCE Manuals and Reports on Engineering Practice No. 106

- ASCE. Buried Flexible Steel Pipe: Design and Structural Analysis. ASCE Manuals and Reports on Engineering Practice No. 119.
- ASCE. Pipe Ramming. ASCE Manuals and Reports on Engineering Practice No. 115.
- ASCE. Pilot Tube and Other Guided Boring Methods. ASCE Manuals and Reports on Engineering Practice No. 133.
- AWWA. Steel Pipe – A Guide for Design and Installation. Manual for Water Supply Practices – M11.
- NASTT. Pipe Jacking Good Practices Guidelines.
- NASTT. Pipe Ramming Good Practices Guidelines.

8. EARTH RETAINING STRUCTURES

8.1 General

This section applies to all earth retaining structure types such as gravity walls, piled walls, cantilever walls, bin walls, anchored walls, Mechanically Stabilized Earth walls (MSE walls), and geosynthetic reinforced soil walls (GRS walls). This section shall apply to all earth retaining structures, whether or not they are located at or in conjunction with a bridge structure. This section does not apply to wing walls cantilevered from the abutment seat or the superstructure end diaphragm, which are addressed in other sections of this document. Additional requirements specific only to GRS and MSE Walls are presented in Section 8.3 (Earth Walls).

The Department has experienced many performance deficiencies with bin walls. These deficiencies have been mainly related to excessive settlement due to poor fill compaction during winter construction. Material selection, construction methodology, and construction monitoring should be given particular attention if winter construction of bin walls is considered. If proper fill compaction cannot be practically achieved, then bin walls should not be considered for winter construction.

The height of any earth retaining structure, or the combined height of multiple earth retaining structures, shall not exceed 8.0 m at any location. The height of the earth retaining structure for this purpose shall be taken as the vertical height from the top of the finished grade in front of the wall to the top edge of the retaining wall or coping.

Earth retaining structures shall be designed so that in the final position, they will be battered back against the retained soil from a vertical plumb line by a ratio of a maximum of 50 vertical units to 1 horizontal unit.

Where settlements are expected, the retaining structures shall be designed to accommodate settlement and allow for reinstating design surfaces. This is a particular consideration at bridge approaches, where fill-around structures are often prone to settlement while bridge abutments on piles are rigid points.

Any bridge components located behind earth retaining structures, such as abutment seats, integral cantilevering wing walls, abutment deck joints, abutment bearings, and traffic barriers, shall be designed to accommodate any movements resulting from lateral wall displacements.

In locations where traffic runs adjacent to the top of, and nominally parallel to, an earth retaining structure, roadway drainage and the allowable encroachment of rain runoff into adjacent traffic lanes shall be accommodated in the same manner as on bridges and in accordance with Section 14 (Bridge Drainage).

8.2 Barriers and Railings on Top of Retaining Structures

Where earth retaining structures are placed within the footprint of a bridge deck and in close proximity to a bridge abutment, a walkway shall be provided between the earth retaining structure and the front of the abutment for inspection purposes. See Section 6.4.2.2 (Inspection Access). The walkway shall include provisions to direct water away from the abutment seat and away from the face of the earth retaining structure towards a swale adjacent to the walkway. The walkway swale shall be detailed so it drains with a minimum 0.5% grade towards a drainage system outside of the bridge abutments.

In locations where traffic runs adjacent to the top of, and nominally parallel to an earth retaining structure, a rigid barrier shall be provided. To determine the appropriate Test Level requirements for the rigid barrier, the earth retaining structure shall be considered to be a structure and the requirements of CHBDC Section 12 (Barriers and highway accessory supports) shall apply. The earth retaining structure shall be designed to fully resist the collision loads applied to the barrier and loads from any attachments such as signs and lamp posts. Approach rail transitions shall be provided at the ends of the rigid barrier.

In locations where a sidewalk or a combined pedestrian/cyclist use pathway runs adjacent to the top of, and nominally parallel to an earth retaining structure, a pedestrian and bicycle barrier shall be provided. The earth retaining structure shall be designed to fully resist the loads applied to the barrier and loads from any attachments such as signs and lamp posts.

At locations where a traffic barrier, pedestrian rail or bicycle barrier is not mounted directly on top of an earth retaining structure, a safety railing shall be provided unless otherwise Approved. The Department will consider the height of the wall, the surface in front of the wall and the potential for people to be in the area when deciding whether Approval is granted to omit the safety railing.

The railing shall be mounted on the top surface of the earth retaining structure and shall be designed as a “guard” in accordance with Part 9 of the current NBCC. The Consultant shall design and detail the safety railing and anchorages on the Detailed Design Drawings.

Safety railings shall have a minimum height of 1070 mm and shall consist of vertical posts with no less than two horizontal rails. Safety railing anchorage assemblies shall be cast into the top concrete surface and shall not be field drilled. All steel components for safety railings shall be galvanized in accordance with ASTM A123 and F2329. All steel components for safety railings shall conform to CSA G40.21 Grade 300W, except that anchor rods conforming to ASTM A307 are also acceptable. Standard chain link fencing may be acceptable in certain locations but must be Approved.

8.3 Earth Walls: MSE and GRS

While ‘mechanically stabilized earth’ is intended as a general description, in this section MSE is intended to refer specifically to wall systems consisting of a rigid structural facing connected to soil anchors (metal ties, geogrid, etc.) embedded in the fill behind the wall. This is in contrast to a geosynthetic reinforced soil (GRS) wall in which the facing elements are typically more flexible and are not considered to be structural/load bearing. These walls are either proprietary designs, with detailed design of individual elements completed by the supplier or designed by the Consultant (or their specialized Geotechnical Sub-Consultant) based on general design parameters provided by the manufacturer of the wall system. ‘Mechanically stabilized earth mass’, ‘reinforced soil mass’ or ‘soil reinforcement zone’ applies to both MSE and GRS walls, and all describe the overall extent of the reinforced earth volume.

The requirements in this section apply to both MSE and GRS walls, except where specifically noted. Additionally, there are requirements in other sections that also apply to MSE and GRS walls - see Section 5 (Durability and Materials), Section 6.4 (Bridge Embankments/Abutments) and Section 8.1 (General).

Walls that are proprietary designs shall meet design-build requirements in SSBC Section 25 (Mechanically stabilized earth walls).

Additional guidance is also available in FHWA publications FWHA-NHI-10-024 and FWHA-NHI-10-025 (Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes - Volumes I and II).

8.3.1 Geometric Requirements

The geometry of the walls, including the associated headslopes and embankments, shall be sloped so that all drainage is directed away from bridge abutments.

Upgraded backfill specifications required for the wall – i.e., material and/or compaction requirements - shall extend a minimum of 0.5 m beyond the end of the soil reinforcement.

Acute corners less than 70 degrees (measured between backfill sides of fascia panels) are not allowed.

8.3.2 Surface Drainage

Highway and bridge surface drainage shall be controlled and channeled away from the back of the walls and the mechanically stabilized earth mass.

Water carrying appurtenances, such as catch basins, drainage inlets/outlets, culverts etc., shall be placed away from or beyond the ends of the soil reinforcement zone, or provisions shall be made to mitigate the detrimental effects of potential leakage.

For walls with steel soil reinforcement, no drain trough or wick drain carrying roadway drainage shall be located over steel soil reinforcement and no drainpipe carrying roadway runoff shall be located within the wall's soil reinforcement. All steel soil reinforcement shall be protected from exposure to roadway de-icing salt by an impermeable geomembrane placed above the top layer of soil reinforcement. This shall include soil reinforcement directly below a roadway as well as immediately adjacent to the roadway for a minimum width of 5 m parallel to the outer edge of the roadway shoulder. For walls that run parallel to the roadway, an impermeable membrane shall be provided to intercept any drainage from the roadway base layer and direct it away from the wall face. The membrane shall be sealed to prevent leakage, slope at a minimum of 5% to drain away from the bridge and wall and be connected to an outlet beyond the reinforced soil mass. A non-woven geotextile filter fabric layer shall be placed below and above the membrane to prevent puncture. In all cases, the geomembrane material shall be made continuous and water-tight and shall extend a minimum of 500 mm beyond the extent of the steel soil reinforcement. Any necessary joints shall be shingled in the direction of drainage and welded or bonded to prevent leakage.

8.3.3 Sub-Surface Drainage

Sub-surface drainage is defined as drainage occurring within the mechanically stabilized earth mass. While it is the MSE or GRS wall supplier's responsibility to incorporate sub-surface drains into their backfill, it is the Consultant's responsibility to determine the expected water level in the mechanically stabilized earth mass and to tie the sub-surface drainage in with the overall site drainage. In most circumstances, it is reasonable to assume that with properly designed backfill and sub-surface drainage, the water level within the mechanically stabilized earth mass will be at the invert level of the weeping drains. If the Consultant determines that a higher water elevation is likely to occur, the expected design water level shall be shown on the Detailed Design Drawings as a loading condition so that the MSE or GRS wall supplier can incorporate it into their design.

As a minimum, continuous weeping drains consisting of perforated 150 mm diameter pipe complete with filter sock shall be provided near the front and the back bottom corners of the mechanically stabilized earth mass. The weeping drains shall be daylighted into drainage ditches or connected to drainage collection lines for positive drainage and disposal. The high end of the drains shall be capped and sealed to prevent the ingress of native or backfill materials.

8.3.4 Waterways, Water Carrying Appurtenances and Utilities

MSE and GRS walls shall not be used within a waterway or floodplain unless the designer can demonstrate to the Department that potential scour and any other deterioration mechanisms will not occur during the design life of the wall. The use of MSE and GRS walls within a waterway or floodplain requires Approval.

MSE or GRS walls shall not be placed over or in the vicinity of any utilities or water carrying appurtenances, unless such utilities or water carrying appurtenances can be removed and repaired without disturbing the mechanically stabilized earth mass, excavation of such utilities or water carrying appurtenances can be executed without impact on wall stability, and agreement is obtained from the utility owners. Water carrying appurtenances include catch basins, drainage inlets/outlets, and culverts. Utilities or water carrying appurtenances carrying potentially eroding materials shall not be permitted within 10 m of any mechanically stabilized earth mass unless the utilities or water carrying appurtenances are sufficiently enclosed within a containment structure that is designed to prevent exposure of any leakage from the utilities or water carrying appurtenances into or onto the wall system, and the extent and design of the containment structure shall be sufficient to protect the wall system against any discharges from this containment structure. No change of direction of utility lines, and no valves, valve chambers or any other discontinuity shall be permitted within the mechanically stabilized earth mass.

8.3.5 Facing

Unless otherwise Approved, all MSE walls shall be faced with precast concrete fascia panels or gabion baskets in accordance with the SSBC. The buried face of MSE wall fascia panels shall be in full contact with compacted backfill.

8.3.6 Coping Cap

A cast-in-place concrete coping cap shall be placed on the top of all MSE walls that have precast concrete panel facings, except for situations where the MSE wall requires a concrete barrier slab. The top of the cast-in-place concrete coping cap shall be smooth, have no steps or abrupt changes in height and shall have a 3% wash slope towards the back of the wall.

Copings shall have control joints and shall have drip grooves in the soffit. Control joints shall be located to line up with the precast concrete fascia panel joints, shall be perpendicular to the wall alignment, and shall in no case exceed 4 m spacing. At control joints, all longitudinal reinforcing in the coping cap shall be discontinuous and shall have a 50 mm cover measured from the center of the control joint.

8.3.7 Barriers/Railings

Barriers and railings on top of walls shall be provided in accordance with Section 8.1 (General). Where rigid bridge barriers are to be located on top of the wall, they shall be supported on moment slabs to resist sliding and overturning. Some guidance is given in Bligh, R., et. al. (2010). TL-4 bridge barriers on top of walls shall be detailed similar to TEC standard drawing *S-1798 (TL-4 Single Slope Concrete and Double Tube Type Barriers along Top of MSE Wall)*.

The Consultant shall fully design and detail all barrier and safety railing components. The Consultant shall show all barrier and safety railing loads on the Detailed Design Drawings so that the wall supplier can incorporate these forces into their wall design.

8.3.8 Vertical Slip Joints

In instances where a continuous length of MSE or GRS wall cannot be built all at the same time as a result of an existing obstruction (e.g., existing bridge, or existing roadway, etc.), the continuous length of wall may be divided into multiple sections and some or all of the sections constructed at separate times. In such instances, if a large differential settlement is expected between any of the various sections of the MSE or GRS wall, appropriately designed full-height vertical slip joints shall be provided between adjacent wall sections.

8.3.9 Obstructions within the Backfill

Obstructions within the mechanically stabilized earth mass, such as foundation piles and associated casings, casings for future pile installations, or other obstructions, shall be accommodated with the appropriate arrangement of soil reinforcing around such obstructions. For MSE walls, the Consultant shall assume that the splay angle of the soil reinforcement cannot exceed 15 degrees from a line perpendicular to the precast concrete fascia panel.

8.3.10 Proprietary MSE or GRS Walls – Consultant Responsibilities

The Consultant's responsibilities for proprietary MSE or GRS walls in accordance with the requirements of Section 20 are summarized as follows:

- Perform geotechnical investigation, short- and long-term global stability analysis, settlement, and movement analysis, and confirm the feasibility of MSE or GRS wall.
- Provide conservative estimates for long-term settlement, rotation, and translation to ensure compatibility with bridge structural components.
- Perform a site survey to establish wall location and layout.
- Prepare 'Wall Layout' Detailed Design Drawings and Special Provisions to include the following information:
    - MSE or GRS wall location, layout, and geometry control.
    - Overall site drainage and sub-surface drainage, including coordination of bridge deck drainage and surface drainage with the MSE or GRS wall components.
    - Allowable bearing capacity, allowable rate of fill placement, and settlement predictions.
    - Barrier and safety railing designs.
    - All loads being exerted onto the MSE or GRS wall components that are not part of the MSE or GRS wall supplier's scope (e.g., loads from roadway barriers, safety railing, attachments, high water level).
- Review design notes and shop drawings submitted by the Contractor to ensure all specification requirements are met. This shall include a review of the Contractor's global stability check using actual soil properties of the backfill used for the project.
- Monitor construction as required, with particular attention to the following quality assurance tasks:

  - Confirm site preparation, site drainage, and wall geometry are in accordance with the design.
  - Confirm fabrication is in accordance with the Contractor's design of the MSE or GRS wall components.
  - Confirm construction of the MSE or GRS wall is in accordance with SSBC Section 25 (Mechanically stabilized earth walls), the Contractor's MSE or GRS wall construction specifications, and the Contractor's MSE or GRS wall design.
- At the end of construction, the Consultant shall update and submit the 'Wall Layout' Detailed Design Drawing to reflect as-built conditions based on the Contractor's as-built drawings and forward all reviewed design notes and shop drawings to the Department for record purposes.

9. BRIDGE BEARINGS

9.1 General

The preferred bearing type in the Northwest Territories is the elastomeric laminated bearing, and it shall be used wherever possible. Where high loads prevent the use of elastomeric laminated bearings, pot bearings are preferred. Continuous plain elastomeric strip bearings are typical under shorter-span precast concrete slabs or side-by-side box girder bridges. Where none of these bearing types are practical, the Consultant shall discuss with the Department to determine the appropriate bearing type (such as spherical bearings) and obtain Approval for an alternative bearing type.

Bearing replacement must be considered in the design. Jacking points, design jacking loads and all steps required in the replacement procedure shall be shown on the design drawings. Sufficient horizontal and vertical space shall be provided between the superstructure and substructure to accommodate the required jacks for replacing the bearings.

To meet the intent of CHBDC Clause 4.4.10.5 (Minimum support length requirements for displacements) the minimum support length/bearing width for sliding bearings along the movement line shall be 200 mm in Seismic Performance Category 1 and 300 mm in Seismic Performance Category 2. Longer support lengths may be required for some structures according to the requirements of CHBDC Clause 4.4.10.5.

Plain unreinforced elastomeric bearings and elastomeric laminated bearings shall be fully designed and detailed by the Consultant on the Detailed Design Drawings. For other bearing types, the bearing components between the sole plate and the base plate shall be designed and detailed by the bearing supplier in conformance with SSBC Section 8 (Bridge bearings) and shall be considered a Proprietary Structure design in accordance with Section 20 ( ) of this document.

Bearings shall be designed to be durable over their entire design life under the climatic conditions of the Northwest Territories. All exposed and embedded steel components of bearings, except stainless steel, shall be protected against corrosion. The corrosion protection system shall be one of the following:

- Hot-dip galvanizing in accordance with ASTM A123.
- Metalizing to AWS C2.23 with a minimum zinc coating thickness of 0.2 mm.
- A coating system that is selected from the MOTT Recognized Product List or the TEC Products List.

Materials for bearings shall meet the requirements of Section 5 (Durability and Materials). All steel-bearing components, except stainless steel, shall be hot-dip galvanized or metalized. Bearings shall be capable of resisting required lateral loads and displacements in combination with the applicable vertical loads and all applicable rotations.

The Northwest Territories have experienced bearings “walking out” from the correct position in both the longitudinal and transverse directions, which may be due to temperature differentials unique in northern climates. The Consultant shall design the bearings with positive restraint measures to keep the bearings in their intended position. This may include keeper bars or other devices in addition to the actual bearing design.

Many components of bridge bearings, particularly sliding surfaces, sealing rings, and elastomers, are sensitive to elevated temperatures that can occur during construction activities such as welding, metalizing, etc. Designers must ensure that welding is possible without damaging these elements (i.e., by providing adequate space for welding, defining appropriate weld procedures, reinstating coatings after damage, etc.).

9.2 Preferred Bearing Types

9.2.1 Plain Unreinforced Elastomeric Bearing Pads

Plain unreinforced elastomeric bearing pads without sliding surfaces shall only be used for support of girders where the total movement range of the bearing in any direction over the life of the bridge does not exceed 25 mm or for temporary erection stage bearings for integral bridge girder supports. Plain bearings shall be used for the Department's standard SL girder bridges (standard drawings S-101 to S-111).

9.2.2 Elastomeric Laminated Bearings

Elastomeric steel reinforced laminated bearings have no moving parts and require little maintenance. Laminated bearings also have significant overload capacity beyond the first signs of distress and generally allow ample time for the identification and repair of problems.

Elastomers are flexible in shear but stiff in bulk compression. When compressed, unconfined elastomeric pads will expand laterally. Layers of steel reinforcing limit the lateral expansion that can occur in elastomeric bearings, increasing the compressive stiffness and strength. Thinner elastomer layers lead to less bulging and increased compressive strength and stiffness, but also higher rotational stiffness. Larger rotations can be accommodated by increasing the elastomer layer thicknesses or the number of elastomer layers. The selection of the number and thickness of elastomer layers is a compromise between the need for compressive strength/stiffness and the need for rotational flexibility.

Laminated bearings can accommodate horizontal displacements through shear deformation and/or through the use of a sliding surface. Shear deformation of the elastomer is the preferred option. Where displacements are too large, displacements may be accommodated through a sliding surface by attaching a PTFE sheet and stainless steel slider to the top of expansion bearings allowing the girder ends to slide across the PTFE surface. The use of this sliding surface requires Approval.

When a sliding surface is used, the following must be considered in the design:

- Friction between the PTFE sheet and slider will still cause some shear deformation in the laminated bearing, which must be considered in the design.
- Rapidly applied movements of the bottom flange, such as those resulting from girder rotation under live load, cause a higher coefficient of friction between the PTFE and stainless steel slider than movements that occur over a longer period of time.

9.2.3 Pot Bearings

Pot bearings shall be used only where elastomeric laminated bearings become too large and difficult to fabricate or cannot be accommodated due to geometric constraints. Pot bearings can accommodate high vertical loads and moderate rotations about any axis. A pot bearing does not permit any horizontal displacements and requires a PTFE and stainless steel sliding surface to accommodate horizontal movements. Lateral guides can be added to the bearing assembly to restrict the direction of movement, but concrete shear blocks are preferred for this purpose.

A pot bearing relies on the total confinement of an elastomeric disc. The disc operates in a nearly hydrostatic state of stress and is rotationally flexible but stiff against changes in volume. The total confinement of the elastomeric disc is essential to the performance of the bearing.

Moderate bearing rotations are accommodated by deformation of the elastomeric disc, whereas large deformations may cause the disc to slip inside the pot and lead to abrasion of the disc. This abrasion is mitigated by ensuring that the inside surfaces of the pot wall and piston are smooth. The rotational limit of a pot bearing is reached when metal components come into contact or when the piston binds with the sidewalls of the pot.

Common problems with pot bearings include leaking of the elastomer and binding of the steel components. Leaking of the elastomer is prevented by a system of sealing rings, of either rectangular or circular section, which is crucial to the satisfactory performance of the bearing. Binding of the steel components is prevented by specifying proper clearances and proper leveling during installation.

Pot bearings require precision fabrication, tight quality control and quality assurance during installation to avoid performance problems.

9.3 General Design Requirements

9.3.1 General

Wherever possible, bearing details shall be in accordance with typical detail drawing *T-002 (Expansion Bearings)*. Deviation from the general design details shown on this drawing requires Approval.

The maximum vertical and horizontal bearing reactions caused by all SLS and ULS load combinations shall be considered. When establishing the horizontal design reactions, the designer shall include anticipated bearing loads at all stages of construction, as well as seismic forces specified in CHBDC Section 4 (Seismic design). Wherever practical, reinforced concrete shear blocks shall be used to transfer lateral loads between the superstructure and the substructure, in accordance with Section 9.3.8 (Lateral Load Transfer between Superstructure and Substructure).

No uplift is permitted at bearings during construction or during service at the Serviceability Limit State 1 (SLS1) without Approval. Where uplift at SLS1 cannot be avoided, a hold-down device shall be provided that is independent of the bearings, accessible for inspection and easily serviceable for repairs. The use of hold-down devices requires Approval.

Several distinct aspects of bridge component fabrication and installation will affect the design rotation of bridge bearings. The three factors that most significantly affect the design tolerance of bearings are the initial set of the bearings with respect to rotation, the girder camber at erection, and the initial set of the bearings with respect to elevation. It should be recognized that rotation capacity demand due to construction misalignment and other uncertainties in many cases, is much larger than the sum of other calculated rotations. As the failure of bearings can lead to considerable damage to a bridge’s structural components and expensive repairs, a generous tolerance should be allowed for. Note it is also difficult to accurately predict long-term movements caused by substructure translation or rotation at the time of design.

9.3.2 Bearings Designed by Consultant

Bearing design shall be in conformance with the CHBDC, the requirements in this section, and the most current edition of SSBC Section 8 (Bridge bearings).

The Consultant shall provide, as a minimum, the following on the Detailed Design Drawings:

- Bearing layout.
- Bearing types and the required number of each type.
- Bearing schedule showing design loads, and translation and rotation requirements, including seismic loading requirements as applicable. A sample bearing schedule is provided below.
- Temperature setting graphs for positioning expansion bearing components according to the girder temperature after girder erection and prior to grouting. For prestressed girders, the setting graph shall include one graph for initial construction, and a second long-term graph with allowances for post-tensioning and long-term movements due to shrinkage and creep.
- Complete detailing of bearings between sole plate and base plate including material specifications and all components.
- Sole plate details including taper dimensions (if applicable) and attachment details.
- Base plate details including anchor rods, bearing attachment details, and grout pad details.
- Concrete shear block details including reinforcement details and/or other details to provide lateral restraint.
- Bearing Setting Elevation Table showing top of sole plate elevations plus two empty rows for bearing height and top of grout pad elevations, to be filled in after bearing heights are obtained from the Contractor.

A sample bearing schedule is given in Table 9-1: Sample Bearing Schedule.

Table 9-1: Sample Bearing Schedule

BEARING SCHEDULE											
Bearing:				**1**		**2**		**3**		**4**	
				Value	**LC[4]**	**Value**	**LC**	**Value**	**LC**	**Value**	**LC**
Design Bearing Reaction (kN)	SLS	Vert.	max.								
			perm.								
			min.								
		Long.									
		Trans.									
	ULS	Vert.	max.								
			perm.								
			min.								
		Long.									
		Trans.									
	FLS[1]	Vert.	live								
Design Bearing Movement[2] (mm)	SLS	Long.									
		Trans.									
Design Bearing Rotation[3] (rad)	SLS	Long.									
		Trans.									
	ULS	Long.									
		Trans.									

NOTES:

1. The component of the vertical reaction at the Fatigue Limit State (FLS) is due to live load only.
2. Design bearing movement shall include the maximum unfactored movements, including post-tensioning shortening and long-term creep, obtained from analysis, plus the additional movement capacity required in the design guidelines.
3. Design bearing rotation shall include the maximum factored rotation obtained from the analysis, plus allowances for uncertainties.
4. For ULS load effects, indicate the governing load case from Table 3.1 of the CHBDC.

9.3.3 Bearings Designed by Supplier

For supplier-designed bearings, the bearing supplier shall carry out the design between the sole plate and the base plate in conformance with the most current edition of the SSBC Section 8 (Bridge bearings) as a Proprietary Structure. Any adjustment required for sole plates and base plates shall be approved by the Consultant.

The Consultant's responsibilities for bearings designed by a bearing supplier are summarized as follows:

- The Consultant shall provide all of the same information on the Detailed Design Drawings as listed under Section 9.3.2 (Bearings Designed by Consultant) with the exception of Item (complete detailing of bearings).
- The Consultant shall review the design notes and shop drawings submitted by the Contractor to ensure they meet all requirements of the Detailed Design Drawings, this document, and SSBC Section 8 (Bridge bearings). This review is for conformance to specification requirements only and is not a design check.
- At the end of the construction, the Consultant shall incorporate changes based on the Contractor's submitted documents into the overall record drawings as required.

Refer also to Section 20 (Proprietary Structures by the Contractor) for further information on the Consultant's responsibilities for Proprietary Structures.

9.3.4 Sole Plates, Base Plates and Grout Pads

The thickness of tapered sole plates shall be sufficient to transfer all loads without any inelastic distortion of the sole plate or the girder bottom flange. The thickness shall also be sufficient to provide threaded holes long enough to develop the full torquing capacity of A325 connecting bolts used to connect the sole plates to the bottom flanges of steel girders.

For precast girders, the attachment of sole plates shall be by welding in the longitudinal direction along the edge of the shoe plate. All galvanizing damaged by welding shall be metallized after welding. The weld design size shall be increased to account for weld contamination effects which occur if the galvanizing is not removed in advance of welding. Transverse welding requiring underhand welding is not permitted. Transverse ends shall be sealed with Sikaflex 1a or an approved equivalent sealant in accordance with typical detail drawing *T-002 (Expansion Bearings)*.

For steel girders, sole plates shall be connected to the bottom flanges with galvanized A325 bolts. Bolted connections shall be designed as slip-critical connections and bolt spacing shall meet sealing requirements – i.e., Clause 11.6.1.4 (Bolted connection) of the CHBDC, paragraph two, which allows for the bolt sealing requirements to not be met in specific instances, shall not apply.

The bolts shall be installed through holes in the girder bottom flange into threaded holes in the sole plate. Girder bottom flanges at bearing connections shall be prime coated all around (bottom, top and edges) with organic zinc epoxy primer meeting the requirements of a Class B coating from the TEC Products List or the MOTT Recognized Products List. The galvanized top surface of the sole plate shall meet the requirements of a Class C surface from CHBDC Table 10.8 (Values of k_s and c_s).

Where used, the thickness and size of the base plate shall be sufficient to distribute loads at all stages of construction, and sufficient to distribute all SLS and ULS loads through the grout pad to the concrete substructure at completion.

Where used, base plates and anchor rods shall be grouted after girders are erected, elevations are checked and confirmed, and before any additional deck dead load is applied. After girder erection and immediately before grouting, the longitudinal location of bearing base plates shall be adjusted in accordance with the bearing setting charts. The bearings should always be above the top of the substructure and the design shall allow for some adjustment of girder elevations if necessary. The underside of galvanized base plates in contact with grout or concrete shall have the contact surfaces protected by a barrier coating in accordance with similar requirements set forth in SSBC Section 12 (Bridgerail).

Where used, shim plates used for shim stacks shall be CSA G40.21 Grade 300W or 350W steel and shall be hot-dip galvanized.

9.3.5 Bearing Setting Plane and Tapered Plates

The bearing setting plane is the plane along which translational movements will occur for a sliding bearing. The setting plane shall be horizontal unless otherwise required and Approved. A non-horizontal bearing setting plane may be required for some types of expansion joints that need the bearing setting plane to be parallel to the roadway grade.

Tapered sole plates are required for laminated bearings and pot bearings to bring the bearing assembly as close as possible to the desired bearing setting plane at full unfactored dead load and at the mean annual temperature, bearing in mind there can be uncertainties in girder cambers and deflections, especially in the case of precast concrete girders. When the design taper is less than 0.003 radians, consideration may be given to eliminating the tapered sole plate and increasing the rotational capacity of the bearing by a corresponding amount instead.

The taper of the sole plate shall account for roadway grade and for elastic rotations due to permanent loads. All other rotations (included time-dependent rotations due to permanent loads) shall be accommodated in the design of the bearing pad.

9.3.6 Horizontal Movement and Sliding Surfaces

Elastomeric bearings designed to accommodate thermal movements through shear deformations shall assume an installation temperature of +20 °C for summer construction and -20 °C for winter construction. Where construction timing is unknown, the bearings shall be designed to accommodate a range of installation temperatures between -20 °C and +20 °C. These temperatures shall be adjusted if a site-specific study demonstrates that different temperatures are more appropriate.

Note that these assumed installation temperatures affect the temperature range the bearings will experience. For bearings with a sliding surface, an increase in the dimensions of that surface may be required. For bearings without a sliding surface, the displacement that has to be accommodated through shearing of the elastomer will increase.

Expansion bearings with a PTFE and stainless steel sliding surface shall be centred at -10 °C, unless a site-specific study demonstrates that a different temperature is more appropriate.

The following types of PTFE shall be assumed to determine the coefficient of friction:

- For laminated bearings, use values for unfilled or dimpled unlubricated PTFE.
- For pot bearings, use values for dimpled lubricated PTFE.
- For lateral guides on pot bearings, use values for filled PTFE.

Expansion bearings shall be designed for all relative horizontal displacements between the superstructure and substructure at the bearing location, whether displacements are accommodated through shearing of the elastomer or by way of a sliding surface. When establishing design movements, all movements described in CHBDC Clause 3.9 (Superimposed deformations) shall be considered, including elastic shortening due to post-tensioning, and long-term movements resulting from creep, shrinkage, relaxation, substructure movement and any other internal or external causes. In addition, expansion bearings shall include an excess travel capacity in each direction in order to accommodate installation tolerances, installation temperature deviations, and unexpected substructure movements. This shall be accomplished by providing sufficient rubber thickness in elastomeric bearings, or where sliding surfaces are used, additional length/width of stainless steel sliding plates beyond that required for the design movements. An additional tolerance in the order of 25 to 50 mm should be added to the longitudinal design movements. An additional tolerance of 10 mm should be added to the lateral design movements, however, bearings with lateral restraint guides do not require the additional lateral movement allowance. These additional tolerances require Approval.

For sliding surfaces, the stainless steel sliding plate shall be AISI Type 304 with No. 8 (0.2 µm) mirror finish, with a minimum thickness of 3.2 mm, and shall be shop welded to the bottom of the sole plate with matching stainless steel electrodes.

9.3.7 Load Bearing Plates - Flatness and Machining Requirements

Steel load bearing plates in contact with each other shall be machined to a surface finish of 6.4 µm and a flatness tolerance of 0.001 × longer plan dimension. Surfaces in contact with an elastomeric pad (except sliding surfaces), grout, or cast-in-place concrete do not require machining. Where required, machining shall be performed prior to hot-dip galvanizing. Where the galvanizing process may cause distortion, metalizing shall be used instead.

9.3.8 Lateral Load Transfer between Superstructure and Substructure

Concrete shear blocks are the Department's preferred means for transferring lateral loads from the superstructure to the substructure and shall be used wherever practical. Anchor rods are not preferred since the rods often get bent or broken and are often difficult to repair or replace due to limited accessibility. Concrete shear blocks provide higher resistance to lateral forces and are typically easy to access for any required repairs. The use of shear blocks also limits the function of bearings to the transfer of gravity loads and rotations only. This not only reduces initial bearing costs by removing lateral guide components but will also increase the bearing service life by eliminating components that are more susceptible to damage. Concrete shear blocks shall be designed in accordance with the details shown on a typical detail drawing *T-002 (Expansion Bearings)*.

Shear block capacity shall be taken at 50% of the nominal shear capacity calculated as interface shear resistance as per Section 8 (Concrete structures) of the CHBDC. The minimum restrainer force shall conform to CHBDC Clause 4.4.10 (Design forces and support lengths).

There may be situations where it is advantageous to use seismic isolation bearings or bearings which are positively attached with pintles or by being vulcanized to the steel bearing plates and transfer lateral loads through the bearing assembly. These systems require Approval.

Concrete shear blocks as shown on typical detail drawing *T-002 (Expansion Bearings)* are only suitable for use with concrete substructures. For structures with steel caps, a steel fabrication bolted to the cap providing lateral support to the girder flange is a more appropriate solution.

9.3.9 Design for Jacking and Bearing Replacement

Bridges and bearings shall be designed and detailed to allow for bearing replacement.

The following assumptions shall be made for bearing replacement provisions unless Approved otherwise:

- All girder lines are simultaneously jacked to avoid damage to the deck, diaphragms, and deck joint components.
- Jacking operations under traffic should be the exception and require Approval.
- The superstructure shall not be raised by more than the vertical recovery of the elastomeric bearing material plus 5 mm.
- After raising the structure, jacks are shimmed around the piston, locked to prevent catastrophic hydraulic failure, or the structure is supported by blocking or other methods. Note that the support system must be capable of accommodating thermal movements and lateral loads in the temporary condition.
- For precast concrete girders with double bearings, the pier diaphragm shall be designed for girder jacking forces.

Locations of jack placement (jacking points), maximum allowable differential movement between girders, and any assumptions in addition to the ones cited above shall be specified on the Detailed Design Drawings.

9.4 Design of Specific Bearing Types

9.4.1 Elastomeric Laminated Bearings

The design of steel reinforced laminated bearings shall meet the requirements of this document, the CHBDC, and SSBC Section 8 (Bridge bearings), and shall incorporate the details shown on typical detail drawing *T-002 (Expansion Bearings)*. The design method in CHBDC-19 is similar to Method B of AASHTO LRFD, whereas the design method in previous editions of CHBDC were similar to Method A of AASHTO LRFD. The current design provisions (AASHTO LRFD Method B) provide a more optimized bearing design and are allowed by the Department.

Elastomeric laminated bearings shall be designed for rotations that take place after the bearings are grouted plus an allowance for uncertainties of 0.005 radians at SLS. Rotations need not be considered at ULS.

Permanent rotations and deformations due to installation and construction activities shall be included in the bearing design.

When selecting the elastomer layer thicknesses and the number of layers, the configuration that produces the smallest overall bearing height while accommodating horizontal movements should be used.

AASHTO LRFD Clause 14.7.6.1 (General) provides guidance on selecting the number and thickness of interior elastomer layers. A minimum shim plate thickness of 3 mm is generally adequate. AASHTO LRFD Clause 14.7.5.3.5 (Reinforcement) provides guidance on checking shim plate thickness.

Notwithstanding CHBDC Clause 11.6.6.1.3 (Elastomers), elastomer materials shall conform to low temperature Grade 5 in accordance with AASHTO M251 (Standard Specification for Plain and Laminated Elastomeric Bridge Bearings) and shall also meet the optional test and acceptance criteria specified in Appendix X1 of AASHTO M251.

The entire bearing assembly shall be replaceable without damage to the structure, without removal of any concrete, welds, or anchorages permanently attached to the structure and without lifting the superstructure more than the vertical recovery of the elastomeric bearing material plus 5 mm. Bearings shall not be recessed into plates that are permanently attached to the structure.

Elastomeric pads shall be restrained in the design position by means of 10 mm high bolted keeper bars, as shown on typical detail drawing *T-002 (Expansion Bearings)*. The typical detail drawing shows a bearing with no masonry plate and keeper bars located on the sole plate above the bearing. The Consultant shall consider whether the addition of a masonry plate below the bearing is required, in which case keeper bars may be provided on the bearing plate. A minimum of one set of keeper bars is required, however, both may be provided if desired.

Preferably, horizontal movement shall be accommodated through shear deformation in the elastomer.

In situations where the elastomeric bearing cannot accommodate horizontal movements through shear deformation and a sliding surface is required, bearings shall allow translation by sliding a stainless steel surface against a mating polytetrafluoroethylene (PTFE) element. The PTFE element shall be a 4.8 mm thick unfilled, unlubricated PTFE sheet. An example of detailing for the sliding surface can be found on TEC typical details drawing *T-1761 (Typical Expansion Bearing Details)*. Elastomeric bearing pads on skewed bridges shall typically be oriented perpendicular to the longitudinal girder axis. When the direction of rotation is uncertain, such as for severe skew or for bridges with stiff concrete diaphragms, round elastomeric bearing pads should be considered.

Field welding adjacent to elastomeric pads requires careful detailing of the bearing assembly to avoid damage to the elastomer. The distance between the weld and the elastomer should be at least 40 mm.

For side-by-side precast concrete box girders that are not designed to be fully monolithic with the abutment, pier, or a concrete end diaphragm, bearing pads shall be installed directly on top of the substructure and steel keeper bars (with corrosion protection) or some other restraint method shall be provided to keep the bearings from moving out of the intended position. For additional information, see Section 10.2.3 (Side-by-side Precast Concrete Girder Bridges).

9.4.2 Plain Unreinforced Elastomeric Bearing Pads

The design of plain unreinforced elastomeric bearing pads (“plain bearing pads”) shall meet the requirements of this document and the CHBDC. Plain bearing pads shall be fully designed and detailed by the Consultant, and all loads, translations and rotations shall be shown on the Detailed Design Drawings.

Plain bearing pads shall only be used to support girders where the total movement range of the bearing in any direction over the life of the bridge does not exceed 25 mm or when used as temporary erection stage bearings for integral bridge girder supports.

For permanent bearings, the bearing support surfaces on the substructure shall be fully detailed with control elevations given on a plan layout. The plain bearing pads shall be designed and detailed so that they have full contact on both the underside of the girders and on the substructure supports with consideration given to girder camber and other geometric conditions.

Temporary erection stage bearings shall not be included in the design of girder-to-substructure connections for dead and live load supports during the in-service life of the bridge.

Plain bearing pads shall be laid directly on top of the substructure and shall be detailed on the Detailed Design Drawings.

Cured elastomeric compounds for plain bearing pads used as permanent bearings shall conform to low temperature Grade 5 in accordance with AASHTO M251 (Standard Specification for Plain and Laminated Elastomeric Bridge Bearings) and shall also meet the optional test and acceptance criteria specified in Appendix X1 of AASHTO M251.

Cured elastomeric compounds for plain bearing pads used as temporary bearings shall conform to low temperature Grade 3, 4 or 5 in accordance with AASHTO M251 (Standard Specification for Plain and Laminated Elastomeric Bridge Bearings) and in accordance with the site-specific requirements for temporary bearings.

9.4.3 Pot Bearings

The design of pot bearings shall meet the requirements of this document, the CHBDC, and SSBC Section 8 (Bridge bearings). Pot bearings are designed by bearing suppliers as Proprietary Structures. The Consultant shall review SSBC Section 8 in order to identify where the SSBC overwrites or supplements CHBDC requirements and ensure that all required aspects are incorporated into the design.

The following design considerations are applicable to the Consultant's bearing design responsibilities as well as to the specific pot bearing components that will be designed by the bearing supplier.

Pot bearings can be installed level or inclined, depending on the orientation requirement for the final sliding surface - see Section 9.3.5 (Bearing Setting Plane and Tapered Plates). The bearing shall be grouted after erection of the girders but before any additional dead load (deck, etc.) is placed on the girders. In both cases, an initial rotation will be forced into the bearing elastomer due to the initial girder camber at the time of erection. This initial rotation will be negative but will be released as the girder end rotates into its final position.

Provision for translation shall be through sliding a stainless steel surface against a mating PTFE element. The metal surface shall be stainless steel and shall extend past the PTFE by at least 25 mm at extremes of movement on each side and shall be positioned above the PTFE element. The translational capacity in an unrestrained direction shall be specified on the Detailed Design Drawings.

Notwithstanding CHBDC Clause 11.6.1.1 (Design considerations), pot bearings shall be designed for all rotations that take place after grouting, plus a fabrication and construction tolerance of 0.005 radians plus an allowance for uncertainties of at least 0.005 radians. These additional rotations are unfactored but shall be considered at SLS and ULS. The rotational capacity about the vertical axis through the centre of the bearing shall be a minimum of ±1 degrees.

The rotational capacity about any horizontal axis shall be specified on the Detailed Design Drawings.

The entire bearing assembly between the sole plate and the base plate shall be replaceable without damage to the structure and without removal of any concrete, welds, or anchorages permanently attached to the structure and without lifting the superstructure more than the vertical recovery of the elastomeric bearing material plus 5 mm. Bearings shall not be recessed into plates permanently attached to the structure.

The average contact pressure at SLS for PTFE sliding surfaces filled with up to 15% mass of glass fibers used to face mating surfaces of guides for lateral restraints shall not exceed 45 MPa.

Pot bearing components shall be metalized or galvanized and attached to galvanized plates by bolting. Surfaces in contact with elastomer shall not be metallized or galvanized.

10. GIRDERS/SUPERSTRUCTURES

10.1 General

10.1.1 Load Path Considerations

The use of single-load path structures is not permitted except for single-lane bridges on low-volume roads and requires Approval.

Slab-on-girder bridge structures with two or more design lanes in accordance with the CHBDC shall have a minimum of three girder lines; whereas single-lane bridges may have a minimum of two girder lines.

Bridges using half-joints (in-span hinges) are not permitted.

10.1.2 Future Rehabilitation

All bridges shall be designed with future rehabilitation in mind.

For bridges with two or more lanes, the superstructure and deck shall be designed to allow for a phased deck rehabilitation or replacement - i.e., it must be possible to keep a minimum of one lane open at all times during the construction work.

The assumed minimum lane width during rehabilitation shall be one 3.7 m wide lane.

If concrete movable barriers are used, a minimum sliding distance of 600 mm shall be provided behind the barrier unless the barrier is fixed to the deck and cannot move.

10.1.3 Girder Layout and Design

Continuous bridges shall have the same number of girders on adjacent spans or segments such that each individual girder line is fully continuous from end to end. Other systems require Approval.

Steel girders are often preferred due to transportation and construction constraints of concrete girders.

Precast prestressed concrete girders and steel girders that are designed to act compositely with the deck for in-service conditions shall be designed so that the non-composite girders can carry the deck dead load and the appropriate construction loads in an unshored condition.

10.2 Precast Prestressed Concrete Girder Bridges

10.2.1 General

The Department does not have a preference for specific precast prestressed concrete girders.

Standard drawings are available for precast prestressed SL girder bridges, similar to the TEC girder drawings in Alberta. The standard drawings comprise girders of 10 m, 12 m, and 14 m in length and 0, 15, and 30 degree skew angles and include substructure and thrie-beam railing details. The standard drawings for precast prestressed SL girder bridges require additional site-specific information to make a drawing set ready for construction and are to be used in conjunction with site-specific information and general arrangement drawings.

The requirements presented in this Section 10.2.1 shall apply to all precast prestressed girder bridges, except for the Department's standard SL girder bridges.

Availability and transportation logistics must be considered when selecting a specific girder. As a result of these considerations, girders are likely to be fabricated in Western Canada, and the standard girders used by the western provinces are the most obvious alternatives to be considered.

In Alberta, TEC prefers the NU girder for medium and long spans, although other girder shapes are sometimes used (e.g., box girders). Shorter spans typically use side-by-side box girders, including the standard SLC and SLW girders (note that the Department has adopted SL girders as their own standard drawings).

The requirements presented in Section 10.2.1 apply to all precast prestressed girder bridges. Additional design requirements are shown in Sections 10.2.2 and 10.2.3 for I-shaped precast prestressed concrete girder bridges and side-by-side precast concrete girder bridges, respectively.

Systems requiring post-tensioning at the construction site require Approval due to the added complexity of required construction methods.

If TEC standard girders are proposed for use, the Consultant shall seek Approval for any conflicts with SBDG requirements. The TEC standard drawings must be incorporated into drawing sets according to NAPEG requirements.

Precast prestressed concrete girders shall be designed to meet the following requirements:

- Consultants shall specify the strand stress "immediately prior to transfer" on the Detailed Design Drawings. This approach clearly identifies that the precast manufacturer is responsible for determining and accounting for all plant-related stressing losses, such as relaxation, bed shortening, abutment tilt, and seating losses. The precast manufacturer can then establish the jacking stress required to provide the specified strand stress immediately prior to transfer. It has been found that the limit of 0.75 f_{pu} for stress immediately prior to transfer is reasonably safe to achieve in most cases. Consultants should check with the precast manufacturer and obtain the correct allowances for prestress losses prior to deciding on the appropriate strand stress value immediately before transfer.
- Significant prestress losses occur in deflected pretensioning strands due to friction from hold-down devices. The Consultant shall understand and incorporate the effects of these prestress losses into their design as appropriate. The Consultant should check with the precast manufacturer to obtain appropriate friction losses and to discuss the appropriate strand stress value immediately prior to transfer. The Consultant shall show the specified stress for the deflected strands on the Detailed Design Drawings and confirm the friction losses during fabrication with the precast manufacturer.
- Skew over 15 degrees shall be avoided where possible and requires Approval. If skews over 30 degrees are Approved and used, sharp corners at ends of girders shall be chamfered as a precaution against breakage. Box girders shall be skewed in increments of 5 degrees.
- To facilitate a reasonable turn-around time for girder casting, the specified concrete strength at release shall not be more than 45 MPa, unless the designer can demonstrate to the Department that higher strengths at

release can be achieved to permit turn-around times from a minimum of two potential precast manufacturers, who typically supply precast girders of that type to the Northwest Territories.

- The minimum age for girders before field cast continuity connection shall be 60 days. Girder and deck design and detailing shall consider the effects of differential camber between girders. Consideration of differential camber is particularly important for bridges with side-by-side girders without overlays or toppings. Side-by-side girders without a topping should not be used where differential camber will affect the durability or functionality of the crossing, including but not limited to rideability or drainage issues.
- Appropriate allowance for girder shortening due to prestress losses (pretension and post-tension) shall be included in the fabricated length of the girders. This value shall be shown on the Detailed Design Drawings.
- Horizontal interface shear design for composite action shall satisfy CHBDC requirements. The longitudinal distribution of shear forces shall be the same as the ULS shear envelope unless a more demanding shear flow condition can exist based on analysis.
- At bearing locations, girders shall have cast-in galvanized shoe plates anchored into the girders by Nelson studs or welded deformed anchor rods. The shoe plate design shall account for the different support conditions at the abutments and piers and shall transfer all vertical and horizontal forces from the girders into the bridge bearings.
- The girder end zones shall be designed to account for the different support conditions at the abutments and piers at all stages of construction, and whether the girder ends will be permanently cast into a concrete diaphragm.
- All miscellaneous steel that is attached to or embedded into girders, and has exposed surfaces, shall be galvanized or stainless. All intermediate steel diaphragms, including all associated plates, washers, and bolts, shall be galvanized or stainless steel.
- Theoretically calculated cambers shall be determined by the Consultant and shown on the Detailed Design Drawings. Camber data shall be provided for each stage of fabrication and construction, including prestress transfer, girder erection, deck pour, post-tensioning (where applicable), superimposed dead loads (SIDL), long-term, etc. Data shall be presented on a camber diagram in the Detailed Design Drawings. An example of a camber diagram is presented in the TEC Engineering Drafting Guidelines for Highway and Bridge Projects Appendix E – Bridge Drawing Checklists. Camber for precast prestressed concrete girders can vary substantially from estimated values due to variations in concrete properties, storage conditions, shrinkage and creep. Proper detailing of stirrup projections, girder end/bearings, and selection of deck haunch thickness are required.

10.2.2 Precast Prestressed Concrete I-shaped Girder Bridges

Precast prestressed concrete I-shaped girders are not common in the Northwest Territories. Further guidance on I-shaped girders, in particular for NU Girders, is provided by TEC through the NU Girder Bridge Design and Detailing Manual (NUGM), TEC standard typical detail drawings *T-1750 through T-1753 (NU Girder Bridges Typical Details - Sheets 1 through 4)*, and the TEC Bridge Structures Design Criteria. If any discrepancies exist between the TEC provided information and the SBDG, the SBDG shall govern.

10.2.3 Side-by-side Precast Concrete Girder Bridges

Side-by-side precast concrete girder bridges are defined as having precast girder units with little or no slab spanning transversely between girder units. In addition to the requirements presented in Section 10.2.1 (General), side-by-side precast girders shall meet the following requirements (not including the Department's standard SL, girder bridges) unless Approved otherwise:

- Side-by-side precast concrete girders deeper than 700 mm shall receive a wearing surface or be fully composite with a reinforced cast-in-place concrete deck, as there is often noticeable differential camber between individual girders. Where an asphalt and membrane wearing surface system is used, drainage provisions shall consider effects of differential camber of adjacent girders in the design to avoid standing water.
- Side-by-side precast concrete girders deeper than 700 mm shall be either fully composite with a reinforced cast-in-place concrete deck or have reinforced shear keys resistant to cracking and deterioration.
- Side-by-side precast concrete box girders that are designed to be fully monolithic with the abutment or pier, may be erected on plain bearing pads on abutments and piers as long as a permanent connection with the substructure is made through cast-in-place concrete diaphragms – between and under girder ends at piers and behind and under girder ends at abutments.
- Side-by-side precast concrete box girders that are not designed to be fully monolithic with the abutment, pier or a concrete end diaphragm shall be supported on a continuous strip bearing or on two elastomeric laminated bearings at each girder end (one bearing nominally under each web). The bearing support surfaces on the substructure shall be fully detailed with control elevations on a plan layout to ensure the support surfaces are configured appropriately, which is particularly important for skewed girders. The difference in elevation between the underside of the girders and the top of the concrete surfaces shall be equal to the thickness of the elastomeric pads.
- Pier diaphragms shall be continuous cast-in-place concrete diaphragms and shall be either pinned, fully monolithic with the pier top or permit free expansion. Positive moment connections of girders over the piers shall consist of either one or a combination of grouted unstressed continuous tendons, lapped and bent-up strands, or lapped cast-in hooked rebar. For side-by-side girders, the minimum separation between girder ends on common supports shall be 400 mm with bent strands or hooked rebar.

10.3 Steel Girder Bridges

10.3.1 General

Bridges designed with welded steel plate girders shall meet the following requirements:

- Welded steel plate girders shall be designed and detailed in accordance with the typical girder details shown on typical detail drawings *T-003* and *T-004 (Steel Plate Girder Bridges – Sheet 1 and Sheet 2)*.
- Vertical stiffeners and girder ends shall be either square to the girder flanges, which are parallel to the road grade, or be true vertical under full dead load. Since the abutment backwall is normally plumb, the abutment detailing shall account for the effects of girder end tilt.
- All welded steel girders, regardless of span, shall be cambered for 100% of dead load deflection and roadway gradeline profile.

- All stiffeners shall be fillet welded to the web.
- All bearing and jacking stiffeners over 20 mm in thickness shall be as follows:

  At locations with negative moment (e.g., interior supports of continuous girders):

  - “fit to bear” to the bottom flange.
  - “fit only” to the top flange.
  - welded at the bottom flange but not at the top flange, unless the bearing or jacking stiffener also acts as a diaphragm stiffener, in which case the top flange shall be welded as well.

  At locations with negligible negative moment (e.g., abutments):

  - “fit to bear” to the bottom flange.
  - “fit only” to the top flange.
  - welded at both the top and bottom flanges.
- Bearing and jacking stiffeners up to 20 mm in thickness shall follow the same requirements given above, except that “fit to bear” may be replaced by “fit only” at all locations.
- “Fit to bear” requires the stiffener surface to be in contact with the inside face of the flange. “Fit only” does not require contact and a space of up to 2 mm is permitted between the stiffener and the flange.
- For long bridges with large expansion movements, the need for multiple vertical bearing stiffeners shall be considered. The design shall ensure that when the bearing shifts away from the bearing stiffener(s), the girder bottom flange does not distort so much as to affect the performance of the sliding bearing.
- Jacking stiffeners for bearing replacement shall be provided on all girders and at all supports. The location of jacking stiffeners shall be based on estimated jack sizes required for bearing replacement, plus sufficient clearance to the edge of the abutment seat or pier cap.
- Diaphragm connector plates (“diaphragm stiffeners”) shall be “fit only” at the top and bottom and shall be welded to both flanges and to the web.
- Intermediate stiffeners in stress reversal regions shall be “fit only” at the top and bottom and shall be welded to both flanges and to the web. Intermediate stiffeners outside of stress reversal regions shall be “fit only” and welded to the compression flange only and cut short of the tension flange with web gap meeting the requirements of CHBDC Clause 10.10.6.4 (Stiffener details at flanges).
- Vertical stiffener plates shall be corner-coped (80 mm vertical x 35 mm horizontal) adjacent to the web. Where stiffener plates project past the outside edge of the flange plates, the projecting stiffener corners shall be coped 45º.
- Longitudinal stiffeners are normally placed on the opposite side of the web to vertical stiffeners. Where horizontal stiffeners and vertical stiffeners intersect on the same side of the web, the horizontal stiffener shall run continuously through a slot in the vertical stiffener. The cut edges of the vertical stiffener at the intersection shall be corner-coped (25 mm x 25 mm) adjacent to the web and be welded to the horizontal stiffener. Similarly, intersecting welds are not permitted between vertical stiffener welds and other web attachments such as gusset plates.
- All weld ends for stiffeners, gusset plates, and other attachments to girders shall terminate 10 mm from the edge or end of plates.

- Gusset plates for the attachment of horizontal bracing shall be bolted and not welded to girders wherever possible. Welded gusset plates for horizontal bracing may be used where there is no risk of fatigue cracking, such as in global compression zones of simple span girders.
- The following features shall be used to prevent staining of substructure concrete:
  - At pier locations, the exterior edge of the bottom flange of exterior steel girders shall have a 19 mm x 19 mm x 8000 mm long rubber drip stop centred over the pier, in accordance with drip stop detail on typical detail drawing *T-004 (Steel Plate Girder Bridges - Sheet 2)*.
  - At abutments, exterior steel girders shall have the same rubber strip attached around the bottom flange at 2000 mm from the face of the abutment walls. Where steel girders are cast into integral abutment diaphragms, a second rubber strip shall be applied all around the bottom flange of all girders immediately in front of the concrete abutment diaphragm face. See rubber drip stop detail on typical detail drawing *T-004 (Steel Plate Girder Bridges - – Sheet 2)*.
- Changes in girder flange width shall be tapered at 2.5 (longitudinal):1 (transverse) or with a 600 mm radius as shown on typical detail drawing *T-004 (Steel Plate Girder Bridges - Sheet 2)*.
- Shear stud projections from the top of the girder flanges into the deck shall meet all CHBDC requirements for stud development and anchorage and ensure full composite action in accordance with design requirements. The design shear stud projection, measured from the underside of the head of the stud to the top of the bottom transverse deck reinforcement, should be designed to be a minimum of 50 mm (25 mm plus a construction tolerance of 25 mm). In some cases, it may be more appropriate to design the girders with partially composite action. One such case is when full-depth precast concrete deck panels are used and making the connection for fully composite action is not feasible. Partially composite action is preferred over non-composite action because it prevents floating decks. Designs with partially composite or non-composite action require Approval.
- At all deck joint locations, the girder shall be protected with an approved bridge coating system in accordance with SSBC Section 6 (Structural steel). Notwithstanding the exceptions listed below, the coating system shall be applied to the bottom flange (underside, top and edges), full height of the web (including any applicable bearing or jacking stiffeners), and to the underside of the top flange, and shall extend longitudinally from the girder end to a distance of 100 mm beyond the bearing sole plate or 100 mm beyond the jacking stiffener, whichever distance is greater. The topcoat colour shall conform to US Federal Standard 595C colour FS30045.

  The following exceptions apply:

  - Where bearing sole plates are galvanized, the faying surface of the underside of the bottom flange in contact with the bearing sole plate shall only receive an organic zinc epoxy primer. The primer shall extend the full width of the flange and 15 mm beyond the projected contact surface of the bearing sole plate in the longitudinal direction.
  - Diaphragms intended only for temporary use, i.e., that will be removed when construction is complete, do not require coating.
  - Faying surfaces of bolted connections shall receive only the organic zinc epoxy primer.
  - Any of the portions of the girder noted above that will be encased in cast-in-place concrete shall be left in the bare steel condition with no coating applied.

- If Approved, coating requirements may be waived where girder ends are fully encased in a concrete end diaphragm which precludes deck run-off from contacting the structural steelwork.

10.3.2 Steel Girder Camber

Camber data shall be shown on a camber diagram at 10th span points, centreline of supports, and centreline of field splices, along with net camber values for individual girder segments between splices. For spans longer than 50 m, data shall be presented at 20th span points. Tabulated data shall include deflections due to girder dead load, deck dead load, superimposed dead loads (e.g., curb/barrier, wearing surface, utilities, etc.), and vertical grade. Deck dead load deflections should include the effects of concrete deck creep and shrinkage.

For complex structures, such as those with long spans, curves, or high skew, more rigorous analysis and camber diagrams for individual girder lines shall be considered.

Camber variations for steel girders are normally minor and should be easily accommodated by adjusting deck formwork elevation and the thickness of the deck haunch on top of the girders.

10.4 Diaphragms

Diaphragms at supports and within each span (intermediate diaphragms) are required for all slab-on-girder bridge structures. Intermediate diaphragms for slab-on-girder bridges with I-shaped precast concrete girders shall have a maximum spacing of 13.0 m. Intermediate diaphragms for slab-on-girder bridges with steel girders shall have a maximum spacing of 8.0 m.

For girder depths less than or equal to 1200 mm, intermediate diaphragms may be C-shape (channel) or W-shape sections of at least 1/3 (and preferably 1/2) of the girder depth. For girders deeper than 1200 mm, intermediate diaphragms shall be X-bracing or K-bracing and top and bottom horizontals shall be provided. Diaphragms at supports must be designed to carry lateral loads down to the substructure and are typically K-bracing, X-bracing or I-shaped girder sections close to the depth of the main girders.

Diaphragms and girders shall be designed for construction loads during deck concrete placement in accordance with the requirements of CHBDC Clause 3.16 (Construction loads and loads on temporary structures) and SSBC Clause 4.10.5 (Deck formwork). The diaphragms provided shall become part of the permanent structure and be left in place for future maintenance, widening, and rehabilitation.

Diaphragms, exterior steel girders, and exterior precast concrete girders carrying deck overhangs shall be checked to ensure sufficient strength and stability to handle concentrated loads from deck finishing machines, work bridges, and loads from temporary walkways outside the edge of the deck slab. Loads assumed for such design shall be based on realistic estimates for each bridge and shown on the Detailed Design Drawings.

In no case shall deck falsework be assumed to carry lateral tension or compression between girder bays. Any lateral loads imposed on the exterior girders must be resisted by a combination of lateral loading on the flange and the intermediate diaphragms. The use of a tension tie that is welded or bolted to the girders, shear studs, or projecting shear reinforcement to resolve the lateral load is not permitted.

Intermediate diaphragms are typically treated as secondary members, however for curved bridges and certain high-skew situations, intermediate diaphragms should be treated as primary members and designed accordingly.

For bridges with I-shaped precast concrete girders with moderate skew, oversized or slotted holes may be used to accommodate moderate differential vertical camber or horizontal sweep between adjacent girders during erection. Oversized or slotted holes shall meet the requirements of CHBDC Clause 10.18 (Splices and connections).

For bridges with small radius curves or high-skew, differential deflection between adjacent girders due to dead load application can be a concern. Steel bridges shall preferably be erected such that girder webs are plumb after all dead loads are applied (referred to as "Full Dead Load Fit"). The Consultant shall assess the bridge to determine whether the skew effects will result in the webs being excessively out-of-plumb. Refer to AASHTO/NSBA G12.1 (Guidelines to Design for Constructability) - Section 1.6 (Differential Deflections). Where applicable, the Consultant shall include the following note on the steel girder Detailed Design Drawings, under the 'ERECTION' heading:

> "CROSS-BRACING SHALL BE DETAILED SUCH THAT GIRDER WEBS ARE PLUMB UNDER FULL DEAD LOADS".

11. DECKS, CURBS & SIDEWALKS

This section provides design information for decks, curbs, and sidewalks. Bridges in the Northwest Territories generally do not require concrete barriers, medians, or median barriers. If required, the geometry and detailing of concrete barriers, medians or median barriers require Approval.

11.1 Decks

Bridge decks shall normally be designed and detailed as full-depth precast concrete panels or cast-in-place concrete deck slabs. All concrete decks shall have a slab thickness no less than 225 mm except for decks that are supported on side-by-side girder units with little or no slab span, in this case the minimum deck thickness shall be 150 mm.

Full-depth precast panels are desirable in remote locations where the availability of cast-in-place concrete is limited. Generally, the availability of cast-in-place concrete on site must be confirmed prior to designing cast-in-place concrete decks. When full-depth precast deck panels are used, shear connectors, transverse joint connections, negative moment over piers, durability, and potential future rehabilitation, replacement and traffic accommodation issues must be fully addressed.

The use of partial depth precast concrete deck panels requires Approval. Where partial depth precast concrete deck panels are used, they shall be considered as a permanent and integral part of the structure, subject to all of the requirements of the SSBC and CHBDC, and not as temporary formwork. Partial depth precast concrete deck panels shall not extend past the exterior girder centerlines. To enhance interface shear robustness, each group of panels shall be confined on the sides and deck ends by reinforced concrete monolithic with the cast-in-place concrete placed above the panels.

The Department has had poor experience with the quality and quality control when using prebagged concrete. Hence, prebagged concrete will only be allowed under unusual circumstances and when Approved. Note that the SSBC also restricts the use of prebagged concrete.

Stay-in-place deck soffit formwork other than partial or full-depth precast concrete panels, including corrugated steel or timber, is not permitted.

Decks shall meet the following requirements:

- Bridge decks shall consist of Class HPC concrete. Reinforcing shall be in accordance with Section 5.8.2 (Reinforcing Steel).
- Bridge decks with no waterproofing system (see Section 5.4 (Bridge Decks, Roof Slabs, Approach Slabs)) shall have a 10 mm allowance (increased thickness) for abrasion. Note that the HPC wearing surface and associated 10 mm abrasion allowance may be the top surface of a precast girder, a full-depth precast deck panel, or a cast-in-place deck slab.
- Where the ACP deck protection and wearing surface system is used, some guidance for detailing can be found on TEC standard drawings *S-1838, S-1839 and S-1840 (Standard Waterproofing System for Deck and Abutments – Sheets 1, 2 and 3)*. Water management on top of the membrane is important to avoid standing water in the asphaltic layer. Hence, bridge deck waterproofing membranes shall include wick drains at low points and along the gutter lines to allow for the controlled drainage of sub-surface water that penetrates

the asphaltic wearing surface, and discharge of the sub-surface discharge at the bridge ends only unless Approved otherwise. Intermediate discharge locations require Approval.

- Concrete decks shall be reinforced with both upper and lower layers of reinforcing except for concrete decks of side-by-side girders that may have a single layer of reinforcing. Each layer of reinforcing shall consist of two reinforcing directions, one direction that can resist longitudinal or primarily longitudinal forces and one direction that can resist transverse or primarily-transverse forces.
- Experience has shown that the reinforcing obtained using the empirical design method in accordance with CHBDC Clause 8.18.3 (Empirical design method) does not always control cracking as well as the requirements in CHBDC Clause 8.12 (Control of cracking). Hence, for bridges with a concrete-wearing surface, the requirements of CHBDC Clause 8.12 shall be met in addition to CHBDC Clause 8.18.3.
- When the empirical deck design method is used, the deck shall have a minimum slab thickness equal to the greater of the girder spacing divided by 15.0 or 225 mm. Note that the use of the empirical design method requires composite action between the slab and girder over the entire girder length.
- CHBDC Clause 5.7.1 (Load effects in deck slabs supported on longitudinal girders) defines bending moments in concrete deck slabs. Any references to CL-625 in this clause shall be replaced with CL-800 and all stipulated forces and moments presented in CHBDC Clause 5.7.1 shall be pro-rata increased by a factor equal to the ratio of 800/625 (1.28).
- Deck reinforcement required to develop the capacity of bridge barriers is site-specific and shall be included in the Detailed Design Drawings.

Full-depth precast decks shall satisfy the following additional requirements:

- Reinforced concrete shear keys may be used without post-tensioning. The shear key design shall account for all force transfer effects through the shear keys. Shear keys made of ultra-high performance concrete (UHPC) can reduce reinforcing requirements in the shear keys and may be used when Approved.
- A minimum specified gap of 25 mm for grout shall be provided under the panels above the supporting beams, including steel girder splice plates. The gap for grout may be omitted for low-volume road bridges when Approved.
- The deck slab shall be fully composite with the supporting beams.

11.2 Curbs

Specific design information related to bridge barriers is provided in Section 13 (Bridge Barriers & Transitions). Standard drawing *S-006 (Barrier and Curb Details)* shall also be consulted.

Curbs shall meet the following requirements:

- Concrete curbs and barriers shall have crack control joints at a maximum spacing of 3.0 m. They shall be centred between bridgerail posts where bridgerail posts are used. Longitudinal reinforcing in the curbs and barriers shall be discontinuous at control joints. Control joints shall extend down to the top of the concrete deck and the sealant in the joint shall be placed and cured prior to the application of any required deck coatings such as waterproofing membrane or silane sealer.

- Curb reinforcement required to develop the capacity of bridgerail post anchors is site-specific and shall be included in the Detailed Design Drawings. Guidance for the design of decks supporting bridgerail posts is available in AASHTO LRFD Bridge Design Specifications Appendix A13.
- The tops of curbs and barriers shall have a wash slope of 3% towards the roadway.
- Concrete paving lips along the edge of ACP are not permitted. This means that the ACP surface must run continuously across the roadway, from face to face of curbs or barriers.

11.3 Sidewalks

If sidewalks and pathways are included on the bridge, they shall be in accordance with the most recent version of the TAC Geometric Design Guide for Canadian Roads and require Approval.

For raised concrete sidewalks, the curb type (barrier or mountable) shall be compatible with roadside safety and barrier performance. For general guidance, refer to the most recent version of the TAC Geometric Design Guide for Canadian Roads.

Drainage of sidewalks must be carefully detailed to avoid issues including but not limited to standing water or ponding, runoff of water into areas that may lead to deterioration of the substructure or girders, excessive water on the roadway, and potential ice formation under freezing conditions.

Sidewalk drainage along and at the ends of the bridge shall be shown on the detailed bridge design drawings.

12. DECK JOINTS

12.1 General

The Consultant is expected to understand the behavior and limitations of deck joints and shall address any potential issues with the Department during the preliminary engineering phase.

As stated in Section 4.5.2.2 (Articulation), new bridge structures shall be fully continuous from end to end, unless the length of structure requires intermediate deck joints.

Joints shall be designed to be durable over their entire design life under the climatic conditions of the Northwest Territories. All exposed and embedded steel components of joints shall be protected against corrosion. Materials for joints shall meet the requirements of Section 5 (Durability and Materials).

Many of the roads in the Northwest Territories are gravel or consist of chipseal pavement. Stones and rocks easily enter unprotected joints and can lead to premature deterioration of the joints, in particular, the puncture of joint seals. Hence, the Consultant must determine appropriate mitigation measures to prevent premature deterioration of the joints. Potential mitigation measures include paving the approaches with an asphaltic concrete wearing surface (ACP) or the use of cover plates. The approach paving is intended to minimize the number of rocks/stones reaching the joint. While this method can be effective, it is not cost-effective to procure ACP in many locations of the Northwest Territories. Installing cover plates reduces the number of gaps where rocks/stones can reach the joint seals but cover plates are often a maintenance problem, particularly with respect to the loosening of bolts. Cover plates must be carefully detailed and require tight fabrication tolerances.

Deck joints shall run continuously across the full width of the deck, considering skew, crossfall and crown of roadway. Exterior bridge barriers and curbs shall have full cover plates on the inside face and across the top. Interior traffic separation barriers shall have full cover plates on both sides and across the top. Cover plates shall be recessed 5 mm from adjacent concrete surfaces. The tops of deck joints across the width of sidewalks or pathways shall have non-slip surfaces (by applying American Safety Technologies AS-2500 or approved equivalent) and detailed to avoid tripping hazards.

Cover plates over joints on bicycle paths or pedestrian walkways greater than 100 mm in width shall be surfaced with a non-skid protective coating or shall have a skid resistant pattern on the cover plates acceptable to the Department.

Maximum permissible deck joint movements shall be based on unfactored values of thermal effects as well as any other long-term movements consistent with the proper functioning of the joint type (e.g., creep and shrinkage).

All joints shall be designed to be compatible with future jacking of the superstructure for bearing replacement. See Section 9.3.9 (Design for Jacking and Bearing Replacement) for more information on jacking requirements for bearing replacement.

Water ingress into the abutment wall backfill or onto the substructure shall be prevented. Water shall not be allowed to contact any part of the superstructure or substructure below the joints. When open joint drainage is used, drain troughs and other drainage hardware are required. Access to the drain trough and other drainage hardware must be provided for inspection and maintenance. Drain troughs and hardware shall be replaceable.

The Consultant shall ensure that sufficient information is provided on the Detailed Design Drawings to address all site-specific requirements. The following are some of the more common details that shall be included on the Detailed Design Drawings:

- Coordination of the deck joint anchorage and the reinforcing in abutment, deck, barriers, etc. This is particularly important for bridges with challenging geometry, such as high-skew.
- Coordination of the longitudinal and lateral movements of the bridge superstructure at the deck joint location. This is a concern for highly skewed and/or curved bridges, long bridges, and wide bridges. Coordination and attention to detailing is required at the barrier cover plates, finger orientation, bearings, and concrete shear blocks.
- The presence of different barrier types or sidewalks.

Any deck joint components in addition to those listed in Clause 6.3.4.1 (Deck joint assembly (extrusion) delivery and installation inspection and test plan) of the SSBC shall be specified in the Special Provisions.

The Department does not have standard joint details at this time. TEC has standard drawings that may be useful references for the Consultant. All proposed joint details require Approval, whether or not they are taken from these suggested references.

12.2 Deck Joint Types

12.2.1 Joint Selection

Where possible, the designer shall endeavour to use strip seal joints for shorter movement ranges and cantilever finger joints for longer ranges. All deck joints, except finger joints, shall be sealed. Unless otherwise Approved, expansion joints shall be designed as finger deck joints when the total movement is larger than 100 mm.

Designers should select joint types with a reliable track record and for which wearable components are accessible and maintainable with minimal disruption to traffic. Proper design and correct installation are key to satisfactory performance.

12.2.2 Strip Seal Joint

The strip seal deck joint is the Department’s preferred deck joint system. The use of this deck joint system will be limited by the movement capacity of the seal, perpendicular and parallel to the joint. For skew bridges, the movement along the longitudinal direction of bridge movement is resolved into the perpendicular and parallel (shearing of the seal) components with respect to the joint axis. The governing movement limit is reached when either one of the component movement ranges exceeds the respective permissible values.

Only neoprene seals shall be used for strip seal deck joints. No other seal materials will be permitted.

The maximum permissible gap perpendicular to the joint shall include all required design movements plus an additional allowance to account for installation tolerance. The maximum permissible shear movements (i.e., movements transverse to the seal) shall be as specified by the seal manufacturer. Fabrication and installation tolerances shall be given on the Detailed Design Drawings.

It should be noted that the normal component of seal movement is set at the time of concreting the joint extrusion in accordance with the temperature setting chart provided on the Detailed Design Drawings. However, the parallel (shear) component of movement is at zero when the seal is installed. Therefore, the maximum permissible movement parallel to the joint must be based on the maximum temperature difference between the temperature at joint installation and the maximum or minimum design temperature. The designer shall consider a range of installation temperatures between -20 °C and +20 °C when assessing joint movements unless the construction schedule is such that the installation temperature is expected to be outside this range. The designer shall also consider the minimum gap required for seal replacement and ensure that replacement is possible at a temperature of +20 °C.

Strip seal deck joints are typically pre-set with a joint gap in the fabrication shop and then clamped with erection angles for shipping. The deck joints shall be re-set during installation based on the effective bridge temperature at the time of installation, which may be assumed to be the mean shade air temperature taken over the previous 48 hours for concrete structures and 24 hours for steel structures.

As discussed in Section 12.1 (General), the seals of strip seal joints are vulnerable to puncture by stones or rocks. Hence, protection of the seals must be considered in the design. When cover plates are used, the cover plates must be carefully detailed to reduce joint maintenance. The use of cantilevered cover plates may be an option to reduce maintenance.

Where a skew angle exceeding 15 degrees is Approved, snowplow guard plates shall be installed for strip seal type joints to prevent snowplow blades from dropping into the joint gap and catching the edge of the joint extrusion. Welded snowplow guard plates shall not be located directly under wheel paths. Refer to TEC standard drawing *S-1811 (Type 1 Strip Seal Deck Joint - Sheet 2).*

For strip seal joints on bridges with an ACP wearing surface, the construction joint for deck joint block-outs shall be within the extent of the deck waterproofing system. This construction joint shall be shown on the Detailed Design Drawings.

The make and model of the joint as well as the extrusion type shall be shown on the record drawings.

12.2.3 Finger Joints

Finger joints are typically used where the bridge movements are too large to use a strip seal deck joint.

Only cantilever finger joints shall be used. Supported finger joints often have durability issues due to difficulties in installing joints to the accuracy required for satisfactory performance and require Approval.

Finger joints shall be designed together with the appropriate drainage details below the joint to deal with debris and run-off water allowed to penetrate the joint.

The maximum permissible movement for a cantilever finger plate deck joint is 300 mm. There are no specific limits for movements parallel to the joint since the fingers must be aligned in the direction of thermal expansion/contraction.

Finger plate deck joint assemblies are pre-set in the fabrication shop with a gap setting corresponding to the anticipated field installation temperature and then clamped with shipping angles for delivery to the construction site. The Contractor is responsible for identifying the anticipated installation temperature based on their construction schedule and advising the fabricator accordingly. At the time of installation, the Consultant will determine whether the temperature is close enough that the joint can be installed as it was set in the fabrication shop or if adjustments are required.

Finger deck joints have tight fabrication and installation tolerances to avoid fingers rubbing against each other and other issues that may impede intended movements or rideability. The Consultant shall describe the required installation process thoroughly on the Detailed Design Drawings or Special Provisions and shall confirm in collaboration with the Department Representative during construction that these requirements are being followed.

Finger joints may be hazardous for pedestrians and cyclists due to the gaps. Cover plates or other devices should be considered when pedestrians and cyclists are crossing the structure; these devices require Approval.

12.2.4 Other Joint Types

Other joint types may be appropriate for some structural situations.

Compression seals are appropriate where only the accommodation of deck rotation is required, and the resulting joint opening and closing are small, on the order of 10 mm or less. While compression seals are easy to install, their service life is generally lower than that of strip seal joints, and leakage and displacement of seals often occur. Compression seals are prone to puncture by rocks and stones and require protection methods similar to strip seal joints.

Where joint openings larger than 300 mm are required, or where other project requirements preclude the use of both strip seal joints and finger joints, other joint types such as modular seal deck joint systems may be appropriate.

Modular seal deck joint systems are often associated with fatigue concerns, premature failure of elements such as seals, springs, and sliders, and difficulty replacing the seals and other wearable components. However, in certain circumstances, modular deck joints may be the best solution. In these circumstances, the success of the modular deck joint will require particular attention to detailing, selection of appropriate seals, and clearly specifying design requirements for the joint supplier to follow. Special provisions detailing these requirements will be needed. Some guidance on modular joints can be found in the Ministry of Ontario Structural Manual (MTO, 2024).

All joint types other than strip seal and cantilever finger joints require Approval.

13. BRIDGE BARRIERS & TRANSITIONS

13.1 General

The Department's preferred barrier system is a double tube railing and transition in accordance with standard drawings *S-004 (TL-4 Double Tube Type Bridgerail, Bridgerail Details) and S-005 (TL-4 Double Tube Type Bridgerail, Approach Rail Transition Details)*. This design shall be used wherever practical up to a TL-4 test level. Other bridge barrier and transition systems must be crash-tested and require Approval.

The Department 's standard thrie-beam barrier (*S-007 (TL-2 Thrie Beam Bridgerail)*) shall be used in combination with the Department's SL girder bridge systems (*S-101 to S-111*) . The use of a thrie-beam barrier in other applications requires Approval.

There may be situations where a different barrier system is warranted, such as the following:

- A very low-volume road may not require a TL-4 barrier and a thrie-beam barrier may be acceptable.
- Where pedestrians and/or cyclists are present.
- Where the combination of traffic volumes and roadway geometry requires a TL-5 barrier.
- Where the barrier acts as a separation between the roadway and sidewalk (as the posts may become a snagging hazard).

Both TEC and MOTT have examples of other barrier systems, including combination barriers, sidewalk fences, and TL-5 barriers. If any of these barrier types are warranted, the Consultant shall review the available standards, propose to the Department the most appropriate barrier system, and obtain Approval.

For all projects, the bridge barrier exposure index shall be calculated in accordance with the CHBDC. Once the barrier exposure index has been calculated, the appropriate barrier test level can be determined from the tables in the CHBDC.

Bridge barrier end transitions provide a gradual change in stiffness from the rigid bridge barrier to the flexible approach guardrail system. Beyond the bridge barrier end transition, the type of approach guardrail and terminal ends are determined by roadway design requirements.

During construction, cast-in-place concrete portions of the curbs are occasionally cast incorrectly, resulting in a curb/barrier that is too high or too low. In some situations, it might be more detrimental to the barrier to try to rectify this than accept the deviation. For the standard Department barrier identified in Section 13.2 (Double Tube Bridge Barrier and Transitions), the as-built vertical dimension of the cast-in-place concrete curb may vary by +/- 25mm from the dimensions shown on the drawings before rectification is required. In addition, the overall height of the barrier (including the steel post and rails where applicable) may vary by +/- 25mm from the dimensions shown on the drawings without further action. If these dimensions vary by more than the specified tolerance, the barrier may still be accepted by the Department based on the Contractor providing an authenticated engineering submission showing how the barrier still meets the CHBDC and the crash-tested requirements for the appropriate barrier type, Approval must be obtained before proceeding in this manner. Note that the specified tolerance applies to gradual changes only, it does not apply to rapid changes resulting in an effective slope greater than 1V:20H.

For situations where traffic barriers need to be supported on a moment slab, e.g. on top of MSE or GRS walls, TEC standard drawing *S-1798 (TL-4 Single Slope Concrete and Double Tube Type Barriers along Top of MSE Wall)* may be consulted for guidance.

TL-5 barriers are only required in exceptional situations in the Northwest Territories, and the use of TL-5 barriers requires Approval. Examples of TL-5 barrier designs are available in other jurisdictions such as the TEC TL-5 double tube type bridgerail as shown on TEC standard drawings *S-1702* through *S-1705 (TL-5 Double Tube Type Bridgerail)*.

13.2 Double Tube Bridge Barrier and Transitions

The Department's standard barrier up to a TL-4 test level is the double tube railing in accordance with standard drawing *S-004 (TL-4 Double Tube Type Bridgerail, Bridgerail Details)*. The approach transition details shown on standard drawing *S-005 (TL-4 Double Tube Type Bridgerail, Approach Rail Transition Details)* shall be used for all roadways where the approach roadway requires up to a TL-3 roadside barrier design. The standard drawings are based on NCHRP 350 (Ross et al. 2010) testing.

Approach roadway warrants higher than TL-3 are only required in exceptional situations in the Northwest Territories. In these cases, the Consultant shall develop a site-specific design and obtain Approval.

13.3 Modifications to Approach Transition Barriers

Beyond the bridge, the Department's standard approach transition shown on standard drawing *S-005 (TL-4 Double Tube Type Bridgerail, Approach Rail Transition Details)* is typically connected to a TL-3 or lower test level approach roadway barrier such as a W-beam guardrail system.

Occasionally, intersections will be located too close to the end of a bridge to allow the full-length approach transition to be installed. In such situations, consideration should be given to using TEC standard drawing *S-1815 (TL-2 W-Beam Guardrail Approach Rail at Intersection Details)*. This is a crash tested TL-2 W-beam design suitable for use at intersecting ramps and street corners and at a design speed ≤ 80 km/hr (there are currently no similar designs that meet TL-3 design requirements).

Special attention and detailing are required at approach transition barrier posts in order to accommodate drain troughs and other drainage features.

The interaction between curb and guardrail shall be designed based on the principles reported by Plaxico, C. et. Al (2005) (NCHRP Report 537), see also TEC Roadside Design Guide for guidance. When a barrier curb runs up to the end of a bridge barrier, it is important to build the end of the bridge barrier straight out to be continuous with the curb face, and not incorporate the large chamfer under the thrie beam connector. See "CURB END DETAIL" on standard drawing *S-004 (TL-4 Double Tube Type Bridgerail, Bridgerail Details)*.

13.4 Pedestrian and Bicycle Barriers

Where pedestrians or bicycles are expected to be on the structure, pedestrian, bicycle, or combination barriers shall be used. The barrier type for pedestrian, bicycle or combination barriers require Approval.

Pedestrian barriers are intended to be used with sidewalks where the volume of bicycle traffic is expected to be low and infrequent, while a bicycle barrier should be used with sidewalks where the volume of bicycle traffic is expected to be significant enough to cause frequent potential conflict with other users.

In some cases, it may be necessary to assess the site and the expected volume of bicycle traffic. In the absence of a detailed assessment, pedestrian barriers should be considered for rural applications where the sidewalk width is less than 2.5 m. For rural sidewalks that are 2.5 m or wider, and for all urban sidewalks, the bicycle barrier should be used. Any sidewalks with a pedestrian barrier where occasional bicycle traffic is expected should include signage requiring cyclists to dismount.

Care shall be given to detailing of the railings and railing ends to mitigate snagging hazards of bicycle handlebars. For both pedestrian and bicycle barriers, the clear spacing between bars shall not exceed 100 mm in accordance with Clause 12.4.5.2 (Geometry) of the CHBDC.

Examples for pedestrian and bicycle barriers can be found in the TEC and MOTT standard drawings.

Combination barriers often consist of concrete parapets with steel railings on top, but concrete parapets may interfere with snow removal. Another approach is to add steel railings on top of the double tube bridge barrier, this may often be the preferred solution. Note that increasing the height of the double tube railing will require the post and post anchorage designs to be reviewed and adjusted as needed. Snagging hazards also need to be addressed.

For situations where an intersection is located close to the end of the bridge, it is important that roadway vehicle drivers can detect pedestrians/cyclists on the pathway that may be crossing the intersecting road. Depending on the geometry of the intersection, the vertical bars on the pedestrian/bicycle barrier can obstruct the vision of drivers, not allowing them to detect the pedestrians/cyclists early enough. In these cases, the Consultant shall consider a site-specific design that considers sight lines approaching the intersection.

13.5 Bridge Barrier Detailing

Where the standard double tube barrier is used, most required design information is provided on the standard barrier drawing. However, certain design information is site-specific and must be provided on the Detailed Design Drawings. Other barrier types, if Approved, must be fully detailed on the Detailed Design Drawings.

Bridge barriers shall be detailed as follows:

- All dimensions for steel barrier/railing layouts shall be given on centreline of bridgerail anchor bolts.
- Barrier/railing expansion joints shall be provided at all deck joint locations. Additional expansion joints shall be provided at a maximum spacing of 45 m.
- The standard barrier drawing shows a standard barrier/railing expansion joint with a gap of 100 mm, and a large expansion joint with a gap of 200 mm. Considering that most bridge abutments tend to move inwards over the life of the bridge, a large expansion joint should be selected when there is potential for the bridgerail joint to jam before the deck joint closes.
- Steel railings/barrier for bridges with curve radius > 600 m can be chorded between field splices. Steel railings/barrier for bridges with curve radius ≤ 600 m shall be curved. In the latter case, the Consultant shall clearly indicate such requirements on the Detailed Design Drawings. Tube sleeves for splices and expansion joints shall be detailed accordingly.
- For attachments mounted on or closely behind bridge barriers (e.g., street light posts, etc.), base plates and anchors shall be grouted and sealed with a penetrating sealer. A minimum 40 mm nominal thickness grout

pad shall be provided under base plates. The grout shall sit in a grout pocket recessed 20 mm into the surface of the structure. The grout pocket shall be 40 mm larger than the base plate around the perimeter.

14. BRIDGE DRAINAGE

Salt contaminated surface drainage shall be contained and controlled so that it does not come into contact with bridge components (e.g., girders, bearings, diaphragms, substructure and earth retaining structures) with the exception of those components that are specifically allowed by the Department to be exposed to salt contaminated water (traffic face and tops of curbs/barriers, bridge deck and roadway surface, deck joints, deck drains, and drain troughs).

The use of deck drains should be minimized where possible. The goal is to restrict surface water runoff to within the width of the shoulders and avoid lane encroachment of runoff. The risks with runoff encroaching into the driving lanes include hydroplaning as well as drivers attempting to avoid the runoff by moving into adjacent lanes. Drainage issues should receive attention at the planning stage when there is an opportunity to optimize bridge geometry.

Any deck drainage that is discharged onto the headslopes or side slopes shall be done in a controlled manner such that erosion of the headslope or side slopes does not occur. This applies at surface water deck drains along the length of the bridge and at the ends of the bridge with deck joints and sub-surface deck drains. It is typically sufficient for the deck drainage to be discharged directly onto concrete or riprap headslope protection. Where it is not possible to drain onto the slope protection, additional measures shall be taken to prevent erosion.

At the ends of bridges, drain trough collectors shall be used to channel water off the bridge and into riprap lined drainage troughs or channels. Drain troughs are required wherever roadway runoff is expected, whether that runoff is coming from the bridge or from the approach roadway. As a result, drain troughs may be required at the high end of the bridge as well as the low end. Drain troughs may also be required at the ends of earth retaining structures where a roadway runs adjacent to the top of an earth retaining structure. Drain troughs shall drain directly down the side slope (i.e., perpendicular to the roadway) and shall extend to the bottom of the roadway approach fills. Drain troughs shall be designed to function as intended for the expected drainage volume and velocity while accommodating differential settlements and other movements between the bridge and the roadway approach fills. Drain troughs may be eliminated if the roadway drainage at the bridge barrier transitions is controlled by curbs and catch basins. Where the drain trough collector connects to the roadway, the top must be formed to provide a drainage path. Note that sand bag riprap is acceptable for drain troughs and shall be in accordance with the SSBC, Section 9, Drain Troughs. To prevent surface erosion, the Department's drain trough preference is riprap lined channels with robust means to guide water from the bridge and/or approach road into the channel. Asphalt or concrete curbs directing water to the drain trough may be suitable if those materials are already being used on the project.

15. UTILITY ACCOMMODATION

15.1 General

Ducts and conduit systems shall be fully detailed on the relevant bridge Detailed Design Drawings (e.g., abutment drawings, pier drawings, deck, and barrier drawings, etc.). Ducts and conduit systems shall be installed in accordance with the following sections. Unless Approved, they shall not be installed within the thickness of the bridge deck, attached to the bridge girders, or attached to the underside of the bridge deck except as noted below.

15.2 Utility Ducts in Curbs or Concrete Barriers

When requested by the Department, the design shall provide one 75 mm nominal outside diameter utility duct on each side of the bridge deck for the future accommodation of telecommunication or power utilities (i.e., telephone, fibre-optic, or streetlighting). The purpose of these ducts is to allow for unforeseen utility needs that are not currently anticipated at the design stage of the project. These utility ducts shall be placed within the bridge curbs/ barriers and shall be terminated in a weatherproof junction box on the outside of the wing walls, near the wing wall ends and close to ground level, where they are not too visible but can be easily accessed without damaging the existing road or bridge. The utility duct termination shall be detailed on the Detailed Design Drawings.

If additional utility ducts are required for the utility needs of the project, they may be placed within the bridge curbs at Approved locations. Where multiple ducts are placed in the same curb, the Consultant must ensure that the strength of the curb has not been compromised.

Waterproof O-ring expansion fittings or liquid-tight flexible conduit fittings shall be provided at all bridge expansion joints and at locations where sidewalk curbs or barriers could undergo rotational settlement (e.g., sidewalk/roadway barrier over corbel supported approach slabs). Loose fit or tape connections are not permitted. At expansion joints, the O-ring expansion fitting shall be located within the deck joint block-out to facilitate future maintenance.

All utility ducts cast into curbs/barriers shall be rigid PVC DB2, meeting the requirements of CSA C22.2 No. 211.1 and in accordance with the rules of the Canadian Electrical Code. Couplings shall be solvent bell ends (SBE). Pull strings shall be installed and secured at each end of all ducts/conduits for future installations.

15.3 Utility Coordination

If applicable, the Consultant shall contact utility companies to identify and locate any utilities that are located within the highway right of way or that could be affected by the project. The Consultant shall coordinate with the Department and utility companies with respect to any utility relocations or accommodations. Utility relocation or accommodation designs are typically done by the utility company and are not the responsibility of the Consultant. Coordination with the utility companies should be initiated early in the project to identify potential impacts so that potential delays can be avoided.

16. OVERHEAD SIGN STRUCTURES

Overhead sign structures, including bridge support or cantilever types, are procured through a proprietary structure design process by the Contractor in accordance with Section 20 (Proprietary Structures by the Contractor) and Sections 24 (Overhead sign structures and panels) and 27 (Design, supply and installation of proprietary works by the Contractor) of the SSBC.

Overhead sign structures shall be designed in accordance with the requirements of AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals. Overhead cantilever sign structures shall be designed for Fatigue Category 1. In addition, due to the difficulty in predicting the dynamic behaviour of overhead cantilever sign structures and determining which overhead sign structure types are susceptible to galloping, the Department requires that all overhead cantilever sign structures shall be designed for galloping loads, without exceptions.

Hand hole requirements may need to be specified in the Special Provisions.

The Consultant's responsibilities for overhead sign structures are summarized as follows:

- The Consultant shall determine placement, clearance requirements, need for barrier protection, and type of structure (bridge support or cantilever type). The Consultant shall prepare a 'General Layout' Detailed Design Drawing for each individual overhead sign structure. TEC typical detail drawing *T-1721 (Typical Overhead Sign Structure General Layout)* may be used for guidance for this drawing.
- Each overhead sign structure is treated as a bridge and tracked by an asset ID number for design, construction, and inspection. This structure classification is used for all sign support structures where the sign is fully or partially hanging over the traffic lanes or road shoulder.
- The Consultant shall review the design notes and shop drawings submitted by the Contractor to confirm that the requirements in Section 20 (Proprietary Structures by the Contractor), and Sections 24 (Overhead sign structures and panels) and 27 (Design, supply and installation of proprietary works by the Contractor) of the SSBC are met.
- At the end of construction, the Consultant shall update the 'General Layout' Detailed Design Drawing(s) to reflect as-built conditions based on the Contractor's submission and forward all supplier design notes, shop drawings, letter of assurance and other submittals to the Department for record keeping purposes.

17. DETAILED DESIGN DRAWINGS

Bridge Detailed Design Drawings shall be prepared in accordance with the Department's *Engineering Drafting Guidelines for Highway and Bridge Projects (Drafting Guidelines).* It is recommended that all engineers working on Department bridge projects be familiar with the *Drafting Guidelines* as this document contains a range of useful information.

Expected construction loads (such as overhang brackets and screed machines) shall be considered in the design and shown on the design drawings.

The use of standard drawings and typical details is required unless otherwise Approved.

18. COST ESTIMATES

The Consultant is responsible for cost estimates during all stages of design to tender. Accurate cost estimates are critical to project success since they allow the Department to make decisions during planning stages, adequately budget for funding the design development and minimize the risk of inadequate funding when bid prices are received on tenders.

Construction cost estimates should consider the special considerations related to remoteness, access requirements, lodging/camp requirements, site conditions, construction methodologies in northern climates, and locally available labour, suitable contractors, equipment, and materials. Construction costs are often significantly higher in the north due to these factors and as well as local market conditions need to be considered to develop realistic construction cost estimates.

The Association for the Advancement of Cost Engineering (AACE International) provides generally accepted industry guidelines for classification of estimates. The Consultant shall revise the cost estimates as the project matures and shall submit the revised estimates to the Department at the completion of each phase in-accordance with Table 18-1: Cost Estimate Classes, the Department's cost estimate classification system and expected accuracy for each class. The Consultant shall update the cost estimates upon achieving project milestones/major deliverables. Noticeable variations should be communicated to the Department including changes of circumstances influencing the project, risks, or complexity levels variations since the previous milestone.

Table 18-1: Cost Estimate Classes

Class	Purpose	Project Phase	Accuracy (%)
A	Pre-Tender Estimate	Tender/RFP	-15 to +20
B	Baseline Budget Approval	Detailed Design	-20 to +30
C	Preliminary Budget Approval	Concept Design/Preliminary Engineering	-30 to +50
D	Options Screening	Planning Study	-50 to +100

All project cost estimates should be developed using the following guiding principles:

- Be prepared by individuals with expertise from appropriate disciplines and individuals with knowledge, skill, and experience in estimating transportation infrastructure projects using industry recognized, repeatable, and defendable practices.
- Be prepared using a high standard of professional and ethical integrity.
- Be based on the best, most complete information available on the project at the time the estimate is being prepared.
- Reflect the entire approved scope of work, including all the elements and activities necessary to successfully complete the project and include all material, labour, equipment, overhead, and administration costs included in the capital cost.

- Include inflationary impacts over a realistic schedule.
- Include contingency reflective of class of estimate, risk, and uncertainty of the project. Contingency is the funding added to a cost estimate to cover potential cost items that are not known at the time the estimate is developed.

Cost estimates should not be prepared under circumstances that may perceived to be in conflict of interest. The Consultant shall not release or disclose the cost estimates outside the Department without prior specific and written permission by the Department.

The cost estimate shall follow the template provided by the Department and shall include an itemized unit price table. Class A and Class B cost estimates should be in the same format and include the same line items that will be included as part of the construction tender bid form.

The Consultant should discuss with the Department if engineering or any cost other than the capital costs should be included in the estimate and what are appropriate assumptions for these other costs. A clear and concise scope statement identifying the in-scope and out-of-scope parameters for the project is a critical component for preparing a cost estimate. Assumptions for the estimate, including but not limited to quantities and background sources for rates, must be clearly documented. The Consultant must consider impacts from working in northern and often remote locations and realistically account for these factors in their cost estimate. Cost estimates should be updated for all project milestones/major deliverables and as updated information becomes available.

MOTT's Project Cost Estimating Guidelines (MOTI 2020) provide guidance and pointers on how to develop cost estimates. The intent is not for the guidelines to be followed to the letter but to give some overall best practices to be considered in the development of cost estimates. These guidelines can be found at the following link:

https://www2.gov.bc.ca/assets/gov/driving-and-transportation/transportation-infrastructure/planning/guidelines/cost_estimating_guidance.pdf

19. CONTRACT PREPARATION

19.1 General

Contract Documents are the key project deliverable at the end of the design phase and provide the basis for the tendering, contract administration, construction inspection, and post construction monitoring phases of a project. The accuracy and relevance of the information provided is critical to the bidding process and to the overall success of the project.

The Consultant typically supports the Department in preparing tender documents including the assembly of contracts. This section provides guidance on typical construction contracts for bridges. The Consultant is responsible to confirm the delivery method, contents of specific contracts, and the responsibility for each section of the contract documents with the Department at the beginning of detailed design. While several delivery methods are possible, this section focuses on the delivery of tender documents for a design-bid-build contract.

The Department is responsible for advertising and opening of all tenders, awarding of all contracts, and preparing the tabulation of tenders.

Wherever possible, the Consultant shall consider the use of local resources and labour as well as training opportunities when preparing construction tender packages.

The formal submission of tender documents from the Consultant to the Department shall include the following:

- Cover letter.
- Special Provisions.
- Detailed Design Drawings.
- Design quantities (if applicable).
- Cost estimate.
- Reference drawings and materials.

Each of the items listed above are discussed further in Section 19.2.2 (Description of Typical Tender Documents).

19.2 Tender Documents

19.2.1 Overview

The Department will issue tender documents as bid opportunities. Tender documents typically consist of the following sections:

- Section I: Notice of Tender
- Section II: Instructions to Bidders
- Section III: Bid Form
- Appendices:
    - Appendix A - List of Bid Documents
    - Appendix B - Business Incentive Policy; Substantiation of BIP Adjustment (Form IB4-P)
    - Appendix C - Local/Northern Employment and Training
    - Appendix D - List of Unit Prices (Unit Price Contracts Only)

  - Appendix E - List of Options & Substitutions Specified by the Owner
  - Appendix F - List of Options Proposed by the Bidder
  - Appendix G - Plant and Equipment List
  - Appendix H - Tender Schedule Form
  - Appendix I – Northern Manufactured Products List
  - Appendix J - Contractor's Certificate of Insurance
  - Appendix K - Construction Contract

19.2.2 Description of Typical Tender Documents

It is the responsibility of the Consultant to support the Department in the preparation of the bid documents. The majority of the bid documents are based on templates provided by the Department and require only minor input from the Consultant.

The Consultant will provide the following portions of the tender documents:

- **Cover Letter:** The cover letter should include a minimum of the following:
  - The name, address and telephone number and signature of the Consultant.
  - The name, title, telephone number, email address of the designer/contact person responsible for the preparation of the Construction Tender Package.
  - The formal description of the project, tender number, highway/control section number, and the structure ID number.
  - A list of all enclosures.
  - The status of all right-of-way/borrow pit negotiations, if applicable.
  - The status of all utility crossing agreements, if applicable.
  - The status of all environmental licenses and permits (e.g., DFO and Navigable Waters Authorizations), if applicable.
  - If the Class A cost estimate varies from the most recent Class B cost estimate by more than 20% (greater or less), the Consultant shall provide a rationale for the variance.
  - A list of all exceptions to these guidelines together with the corresponding Department Approvals.
- **Special Provisions:** For bridge construction projects, the specifications shall be based on the SSBC and SSHC which are publicly available on the Department's website. Where a project includes work that is not covered by these standard documents, a Special Provision is required. In addition, the Department standard documents refer to project-specific information that is to be included in the Special Provisions; see Clause 1.4.3 (Turbidity Monitoring and Testing) or Clause 3.3.3.1 (Pile Driving Set Criteria and Tip Elevation Requirements) for some examples of when a Special Provision is required.

  Non-technical Special Provisions (i.e., Special Provisions relating to contractual or administrative matters) will be prepared by the Consultant in cooperation with the Department. See Section 19.2.4 (Specific Considerations for Special Provisions) for additional guidance regarding the Special Provisions.
- **Detailed Design Drawings:** The Consultant is responsible for preparing the drawings. The drawings shall meet the following requirements:

  - Provide clean, comprehensive, and 100% complete information for tender and construction in accordance with the Department's *Engineering Drafting Guidelines for Highway and Bridge Projects (Drafting Guidelines)*.
  - Establish Survey Control Points and Working Points as required.
  - Provide a full-sized drawing package (Arch D 24"x36") in electronic format. To facilitate printing, all drawings shall be converted to PDF format using black and white settings. Do not use colour or grey scale settings. Notwithstanding the preceding requirement, Consultant logos may be left in colour if desired.
  - Provide AutoCAD files (*.dwg) in accordance with the bridge CAD standards.
  - Identify Issued for Tender (IFT) set with "Issued for Tender" in the title block and Revision "0". Tender drawing set shall be signed and sealed by a Professional Engineer.
  - After tender award, provide an Issued for Construction (IFC) set with "Issued for Construction" in the title block. IFC drawing set shall be signed and sealed by a Professional Engineer.

- **Appendix D, List of Unit Prices:** For unit price contracts only, the Consultant shall be responsible for preparing the List of Unit Prices to be included in Appendix D of the tender, following the template provided by the Department.

  Each pay item within the list shall include a quantity and shall be associated with a Measurement and Payment clause identifying how the item is to be paid. Pay items that are not included in the SSBC require the Consultant to prepare a Special Provision providing that information, in addition to any technical information required to complete the work.

  The list shall be submitted in both PDF and Microsoft Excel format.

- **Cost estimate:** The Consultant shall provide a Class A cost estimate for the complete project in both PDF and Microsoft Excel format. For unit price contracts, the estimate shall be consistent with the List of Unit Prices included in Appendix D of the tender.
- **Reference drawings and materials:** Reference documents may include drawings, inspection reports, permits, technical reports (e.g., geotechnical investigations), environmental reports, and any other information deemed relevant to the contract. The list of documents to be included is determined by the Department based on the recommendations of the Consultant. Once the list of documents has been agreed upon, the Consultant is responsible for assembling the various documents (whether such documents are prepared by the Consultant or by others) and submitting them in PDF format ready for inclusion in the tender documents.

19.2.3 Submission Format

The Consultant shall supply tender documents as both Portable Document Format (PDF) files and native Microsoft Word (docx) or Microsoft Excel (xlsx) files, as applicable.

Any permits or authorizations, contract drawings, pit plans or other images which are scanned for incorporation in the tender document shall be scanned using the following settings:

- Save the image in GIF, JPG, or TIF format.

- Resolution: 300 dpi.
- Image type: line art or black and white drawing, as applicable.
  Do not use photo, color, or grey scale settings.
- All scanned images shall be easily legible when printed in the proper size format.
- Plans and drawings that are converted directly from CAD format shall be in black and white format only. Do not use color or grey scale settings.

Wherever possible, the Consultant shall consider the use of local resources and labour as well as training opportunities when preparing construction tender packages.

19.2.4 Specific Considerations for Special Provisions

The following points shall be considered when compiling the Special Provisions, in addition to requirements given elsewhere in this document and in the SSBC. The list below is not intended to be exhaustive; the Consultant is to review the SSBC for all locations where 'Special Provisions' is mentioned, to ensure that all necessary provisions are included.

- Special Provisions are required for all site-specific items. This includes but is not limited to items such as traffic accommodation, utilities, unusual hazards, work by others in the project area, Department-supplied materials, environmental requirements, work at night, working near communities etc.
- The special provisions must include all non-standard payment items consistent with Appendix D, List of Unit Prices.
- Quality management 'Hold Points' and 'Witness Points' not already included in the SSBC should be defined in the Special Provisions. **Hold Points** are defined as verification points beyond which work by the Contractor may not proceed until the Contractor notifies the Department and the Department performs mandatory verification activities before giving notice to the Contractor to proceed. **Witness Points** are identified points in the process where the Contractor provides notice to the Department and the Department may elect to review, witness, or inspect relevant aspects of the work. The Contractor is not required to suspend activities relating to a witness point, provided that the required notice has been provided.
- The SSBC refers in several locations to the "closest Environment Canada Meteorological Station or a meteorological station specified in the Special Provisions". Where the closest Environment Canada Meteorological Station is located away from the construction site and may not adequately represent the weather conditions on site, a different weather station may be specified in the Special Provisions, subject to Approval.
- Turbidity monitoring and testing, if required and as referenced in Clause 1.4.3 (Turbidity monitoring and testing) in the SSBC, shall be included in the Special Provisions.
- Pile driving criteria, set criteria and tip elevation requirements not explicitly identified in Clause 3.3.3.1 (Pile Driving Set Criteria and Tip Elevation Requirements) of the SSBC shall be included in the Special Provisions.
- Structural steel items not explicitly identified in Clause 6.1 (General) of the SSBC shall be included in the Special Provisions.

- Miscellaneous components associated with precast girders but not explicitly named in Clause 7.1 (General) of the SSBC shall be included in the Special Provisions.
- Miscellaneous iron items not explicitly named in Clause 13.1 (General) of the SSBC shall be included in the Special Provisions.
- Section 21 (Demolition, disposal and salvage of transportation structures) of the SSBC discusses demolition and removal and includes references for additional information, such as locations for salvaged materials, to be provided in the Special Provisions.
- The removal of temporary works including but not limited to berms, cofferdams and dykes requires site-specific Special Provisions.
- Any site-specific items required for the erection, assembly, and installation of steel or precast concrete girders and other structural steel or prefabricated concrete members shall be defined in the Special Provisions.
- The SSBC are written primarily for new construction projects; rehabilitation projects are likely to require Special Provisions in addition to the SSBC. For example, deck overlay specifications in Section 20 (Deck overlays and rehabilitation of concrete components) of the SSBC will require supplemental information in the Special Provisions when used with rehabilitation projects.
- Coating contracts require specialist knowledge, preferably an AMPP/NACE certified inspector/engineer, with respect to technical Special Provisions as well as construction load considerations related to any temporary enclosures that may be required. The Consultant shall engage coating and hazardous material specialists as required to produce such Special Provisions.

19.3 Addenda and Services during Tender Period

19.3.1 Consultant Services during Tender Period

Inquiries during the tender period will be received by Procurement Shared Services (PSS), distributed to the Department, and then forwarded to the Consultant. The Consultant's response will follow the reverse process and responses to potential bidders are provided by PSS. The Consultant will not communicate directly with any bidders during the bid period. The Consultant must not provide information to individual bidders that is contrary or supplemental to that contained within the tender documents. In instances where supplemental information needs to be disseminated to all plan holders, the Department will issue an addendum.

When a pre-tender meeting is requested by the Department, the Consultant is responsible for scheduling the meeting to take place during the tendering period, once the Department has advised that a tender has been scheduled for advertising. The Consultant shall provide this information to the Department prior to advertising so it may be included in the tender documents.

19.3.2 Addenda

Addenda are revisions to the tender documents that are issued to all known prospective bidders during the tender period. The Consultant shall assist the Department with issuing addenda. The necessity for an addendum will be identified either by the Consultant or the Department, following which the Consultant is responsible for preparing the content of the addendum and providing it to the Department in a timely manner. Wherever possible, the addendum shall be submitted to the Department on the same working day that the need is identified.

If an addendum is submitted to Department close to the scheduled close of tenders, the addendum may require the inclusion of an extension to the closing date to allow all bidders the opportunity to thoroughly evaluate the addendum content.

The addendum submission to the Department shall be in Microsoft Word format. If the addendum includes revised drawings, the drawings shall be provided in PDF format. If an addendum will result in a change to the estimated cost of the project, the Consultant shall provide a revised cost estimate with the addendum submission.

19.3.3 Tender Close/Award

It is the Consultant's responsibility to assist the Department with its evaluation of tender bids, preparing a comparative bid analysis, and preparing the award recommendation letter for the signing authority. Sometimes, the Consultant may be required to carry out an item-by-item review of bid items to better understand bid results. The Consultant typically also reviews other bid documents such as the Plant and Equipment List and the Tender Schedule Form and provides comments to the Department.

20. PROPRIETARY STRUCTURES BY THE CONTRACTOR

20.1 General

This section provides guidance on responsibilities of all parties for delivering proprietary structures in construction contracts and shall be read in conjunction with Section 27 (Design, supply and installation of proprietary works by the Contractor) of the SSBC.

20.2 Definitions

Proprietary structure: Proprietary structures are both designed and built by the Contractor under a construction contract. Examples of proprietary structures include culverts, MSE and GRS walls (except for global stability) and sign structures.

Supplier: Supplier of a proprietary structure. Suppliers will typically be engaged by the Contractor under the Contract.

20.3 Responsibilities

It is the responsibility of the Consultant as part of the design process to determine if proprietary structures are appropriate for a project and confirm this with the Department. The following describes the responsibility of each party during the Proprietary Structure design process.

The Consultant is responsible for the following:

- Retain the overall responsibility as Engineer-of-Record for the project.
- Investigate if proprietary structures are appropriate for the project and recommend proprietary structures to the Department where appropriate.
- Complete the design criteria and terms of reference and prepare tender and contract documents, including required General Arrangement and other drawings and specifications for the design, construction, and installation of proprietary structures.
- Determine and take responsibility for the global stability of completed proprietary wall structures, as applicable.
- Determine and identify all external loads on the proprietary structures.
- Communicate with the Suppliers during the design phase and address any design issues that may arise, review construction/installation specifications and determine supply and construction costs for use in cost estimates.
- Review the Contractor’s submissions for design, as applicable, to ensure that all of the project criteria and designer requirements are met.
- Manage and implement all Department quality assurance and quality auditing during the design, fabrication, construction, and installation of the proprietary structures in discussions with the Department.
- Incorporate the Contractor’s Proprietary Structure Design Report, Letters of Assurance and Record Drawings for the accepted proprietary structure into the final project records.

- Prepare Record Drawings from as-built information provided by the Contractor at the end of the project.

The responsibilities of the Supplier are as follows, refer also to Section 27 of the SSBC:

- Design and supply of the proprietary structures in accordance with the Consultant's design criteria, terms of reference and all related Contract requirements.
- Manage and implement all quality control during the design, fabrication, construction, and installation of the proprietary structure.
- Provide a Proprietary Structure Design Report, Letters of Assurance and Proprietary Structure Record Drawings for all proprietary structures to the Contractor for submission to the Department.
- Complete field reviews and inspections of the proprietary structure throughout the project.

The responsibilities of the Contractor are as follows, refer also to Section 27 of the SSBC:

- Supply and construction of the proprietary structures in accordance with the contract documents and Supplier's design and installation requirements.
- Provide a Proprietary Structure Design Report, Letters of Assurance and Record Drawings for all proprietary structures.
- Overall Quality Management of the construction contract.

20.4 Design Process

20.4.1 Overview of Process

All work identified in the design process shall be performed by the Consultant, unless otherwise noted. The objectives of the design process are to:

- Achieve uniform design and construction standards, such as factors of safety and service life, for all proprietary structures.
- Ensure that all proprietary structures included in the contract documents are acceptable in terms of lifecycle cost and design constraints, including but not limited to structural, hydrotechnical, geotechnical, environmental, and aesthetics.
- Select/specify the lowest installed capital cost structure available at the time of tendering that meets the established design criteria, construction standards and site requirements,
- Allow for fair and competitive selection of available proprietary systems.
- Establish engineering requirements and design and construction responsibilities within the project related to the use of proprietary structures.

20.4.2 Key Deliverables

The key deliverable of the design process consists of a technical memorandum or design report prepared by the Consultant with recommendations regarding use of proprietary structures for the project.

At the tender stage, deliverables consist of all documents necessary to tender the contract, including all drawings and Special Provisions required to define the proprietary structure, design criteria, construction standards, installation requirements, and any other project specific requirements necessary to allow the Contractor to bid, design, and build the proprietary structure.

Key deliverables during the construction phase consist of:

- A Proprietary Structure Design Report, prepared by the Supplier and provided by the Contractor to the Department, including design calculations, specifications, and drawings.
- Letters of Assurance, provided by a professional engineer representing the Supplier and provided by the Contractor to the Department.
- Proprietary Structure Record Drawings, prepared by the Supplier based on as-built details provided by the Contractor.
- Project Record Drawings, prepared by the Consultant based on as-built information obtained from the Contractor, including the Proprietary Structure Record Drawings.

20.4.3 Design Process Stages

Generally, the proprietary structure design process consists of the following four stages:

Stage 1 – Option Development

The Consultant shall investigate different options for situations where proprietary structures may be appropriate and make recommendations to the Department. Tasks under the options development phase typically include:

- Investigate and assess the site.
- Review all available information such as historical data, environmental issues, geotechnical, topographical, debris hazards, etc.
- Establish and confirm design criteria with the Department.
- Conduct further geotechnical investigations, as required, to collect all necessary information.
- Determine general requirements for a structure that might be suitable at the site.
- Coordinate with Suppliers to obtain information about their products to determine suitability for the project.
- Determine durability requirements for the site.
- Conduct a cost/benefit analysis of structural options.
- Provide a recommendation to the Department on the use of proprietary structures that can achieve economic or other advantages.
- Pursue other options as requested by the Department should a proprietary structure be considered unsuitable for the Project requirements.

The Consultant shall summarize their findings in a design report or technical memorandum.

The Consultant shall identify and consult with potential Suppliers and confirm potential Suppliers with the Department. The TEC Products List and the MOTT Recognized Products List may be used to identify potential Suppliers based on compatibility with the project design requirements.

Stage 2 - Prepare Proprietary Structure Design Requirements

The Consultant shall liaise with potential Suppliers to discuss and clarify design requirements including, but not limited to:

- General arrangement drawings.
- Design criteria.
- Durability requirements.
- Project constraints, including alignments, geometrics, geotechnical, hydrotechnical, environmental, construction staging, aesthetics, etc.

Early in the detailed design process, the Consultant shall identify and prepare the following:

- General arrangement drawings that clearly identify the structural requirements including:
    - Structure geometry in terms of spans, lengths, heights, and clearances, etc.
    - All constraints, including alignment, geometry, geotechnical, structural, environmental, construction staging, aesthetic etc.
    - Limits of the pay item associated with the proprietary structure.
    - Design criteria.
- Design criteria that provide the proprietary structure Suppliers with all relevant information required to complete the design of the proprietary structures, including but not limited to:
    - Design codes.
    - Design service life.
    - Aesthetic requirements for wall facing and coping.
    - Wall facing types (e.g., gabions, concrete, concrete block, wire).
    - Fencing requirements.
    - Superimposed loads.
    - Required design methodologies.
    - Bracing limitations.
    - Geotechnical design parameters include allowable soil bearing capacity, setbacks from slopes, anticipated settlements, minimum depth of bury for footings, unit weights and soil friction angles of foundation and backfill soils and required factors of safety.
    - Correct side slopes at earth retaining structures especially where structure is skewed relative to the highway alignment.

  - Structural backfill material meeting the requirements of the SSBC shall be used unless an alternate material of a specific quality and type is required for the performance of the proprietary structure. Fabrication for precast and pre-stressed concrete shall be in accordance with the SSBC as applicable. Any additional requirements shall be specified by the Consultant in the Special Provisions.
  - Any other relevant information.

The Consultant should use this information to liaise with Suppliers to assure that Suppliers can meet the project requirements, discuss availability and costs, and obtain other input required to finalize detailed design. The Consultant shall confirm with the Department that an adequate number of Suppliers are able to deliver the proprietary structure under the Contract.

Stage 3 - Detailed Design and Preparation of Construction Contract Documents

The Consultant shall analyze and compile information gathered in the previous stages to establish final design criteria, determine the final list of proprietary structures acceptable for the project, and determine construction standards and site requirements that best meet project requirements.

The Consultant shall prepare the following contract documentation:

- Special Provisions which include a clear definition of work to be completed within the pay item associated with the proprietary structure.
- General Arrangement Drawings and additional drawings as required that clearly depict the proprietary structure and any special features such as drainage, coping or fencing details, etc. as well as the limits of payment for the associated pay item.
- Approximate Quantities and Unit Prices; the formal contract document package will indicate Lump Sum prices for various elements of the work; for cost estimating purposes the Consultant shall provide estimated quantities and unit prices, along with quantities and unit prices for work related to the proprietary structure which are to be paid for under other payment items.

The Consultant shall assist the Department in preparing addenda and responding to queries regarding the proprietary structures during the tender period.

Stage 4 - Requirements during Construction

- The Consultant shall assist the Department in preparing change orders and responding to inquiries regarding the proprietary structure during the construction period.
- The Contractor shall submit the final Proprietary Structure Design Report containing complete design calculations, specifications, and drawings, sealed by the Engineer of Record for the proprietary structure.
- The Department will distribute copies for review, to the Consultant and others.
- The Consultant shall review the Final Proprietary Structure Design Report for compliance with the contract requirements and consult with other team members to ensure that all of the project criteria have been met.

- If the Consultant does not accept the Final Proprietary Structure Design Report, the Consultant shall consolidate all review comments into a single document and return the marked-up submission to the Department.
- The Department will forward the marked-up submission to the Contractor for modification.
- The Contractor will resubmit a revised Final Proprietary Structure Design Report to the Department for further distribution and review.
- Once there are no exceptions taken to the submission, the Consultant shall accept the proprietary structure design and provide the accepted Final Proprietary Structure Design Report to the Department.
- The Department will forward the accepted submission to the Contractor and to other team members as required.

For Supplier-prepared designs, the Consultant shall conduct field reviews during the construction phase as required. The Supplier will also conduct field reviews and provide Letters of Assurance in accordance with Section 27 of the SSBC. Requirements for field reviews and the Letters of Assurance forms are available on the Department's website:

https://www.inf.gov.nt.ca/en/resources?search_api_views_fulltext=assurance&sort_by=field_resource_publication_date&sort_order=DESC

The Supplier will prepare Proprietary Structure Record Drawings of the completed installation and Letters of Assurance for design and field reviews, sealed by a professional engineer. The Contractor will review the documentation and work with the Supplier to confirm that all information provided is correct and complete in accordance with Section 27 of the SSBC. The Contractor will submit the reviewed package to the Department.

Following construction completion, the Contractor will provide the Consultant with a set of drawings marked up with the as-built information for the proprietary structure along with the Supplier's Letters of Assurance and Proprietary Structure Record Drawings prepared by the Supplier. The Consultant shall review and compile the information and incorporate this information into the project Record Drawings, as applicable.

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APPENDIX A PERMIT VEHICLE CONFIGURATIONS

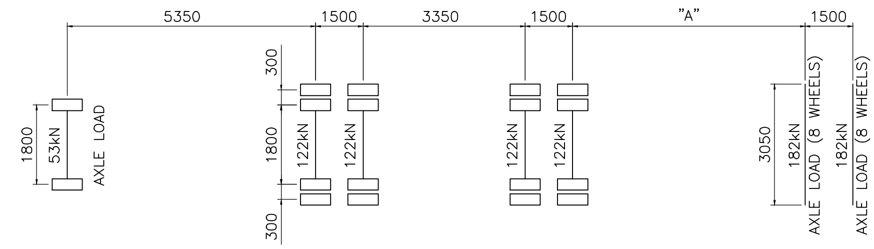
PERMIT TRAFFIC EVALUATION

1. PERMIT TRUCKS #1 AND #2 SHOWN BELOW ARE CLASSIFIED AS PERMIT – SINGLE TRIP (PS) VEHICLES IN ACCORDANCE WITH CSA S6 CLAUSE 14.9.2 (PERMIT – VEHICLE LOADS). THE VEHICLE SPEED SHALL BE ASSUMED TO BE GREATER THAN 40 km/h.

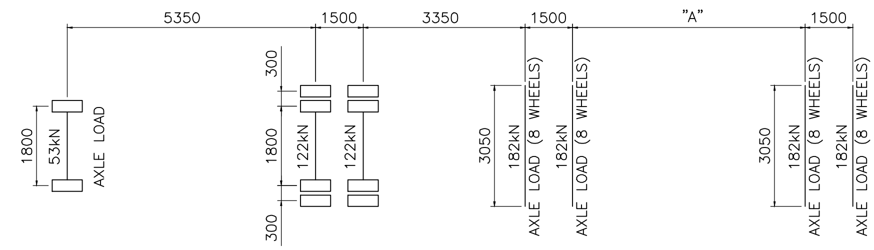
2. PERMIT TRUCK #3 SHOWN BELOW IS CLASSIFIED AS PERMIT – CONTROLLED (PC) VEHICLE IN ACCORDANCE WITH CSA S6 CLAUSE 14.9.2 (PERMIT – VEHICLE LOADS). THE TRUCK SHALL BE ASSUMED TO TRAVEL DOWN THE CENTRELINE OF THE BRIDGE (WITHIN A TOLERANCE OF ±600 mm) AT A VEHICLE SPEED GREATER THAN 10 km/h AND LESS THAN OR EQUAL TO 25 km/h. NO OTHER TRAFFIC NEEDS TO BE CONSIDERED TO BE ON THE BRIDGE OTHER THAN THE PERMIT TRUCK.

3. THE DYNAMIC LOAD ALLOWANCE FOR THE PERMIT VEHICLES SHALL BE FOR IN ACCORDANCE WITH CSA S6 CLAUSE (DYNAMIC LOAD ALLOWANCE FOR PERMIT VEHICLE LOADS AND ALTERNATIVE LOADING).

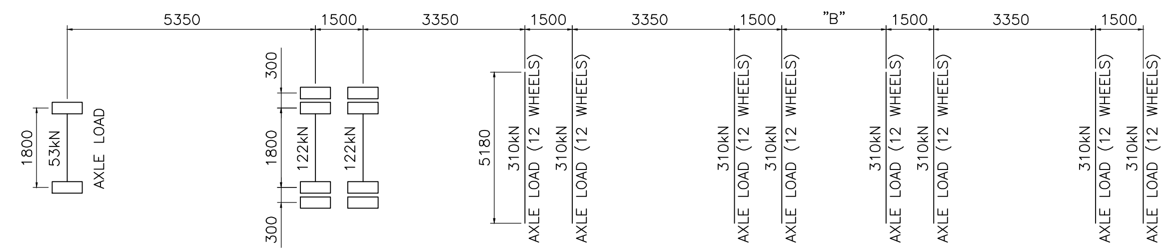
TRUCK #1 – TRACTOR + 8 +16 WHEELS



TRUCK #2 – TRACTOR + 32 WHEELS



TRUCK #3 – TRACTOR + 96 WHEELS



DIMENSION	RANGE
"A"	7.0 m TO 33.2 m
"B"	9.0 m TO 39.3 m

NOTE – LOADS GIVEN ARE SERVICE AXLE LOADS, EXCLUSIVE OF IMPACT FACTOR.