THE CITY OF EDMONTON
DESIGN-BUILD AGREEMENT
CAPITAL LINE SOUTH LRT EXTENSION PHASE 1

Schedule 5 – D&C Performance Requirements

Part 4: Transportation Structures and Building Structures

[NTD: Schedule 5 D & C Performance Requirements – all parts – will be amended July 30 2024 to reflect requirements associated with Appendix A - Affordability Opportunities Amendment Term Sheet]

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PART 4: TRANSPORTATION STRUCTURES AND BUILDING STRUCTURES

SECTION 4-1 – DESCRIPTION OF INFRASTRUCTURE

4-1.1 SCOPE

A. Part 4 [*Transportation Structures and Building Structures*] sets out the structural design and construction requirements for all Transportation Structures and Building Structures unless otherwise specified.

B. The requirements of the City of Edmonton HFDG must be met in their entirety for the design of Transportation Structures and Building Structures.

C. Part 4 [*Transportation Structures and Building Structures*] sets out the design and construction requirements for all Building Structures in conjunction with Part 5 [*Facilities*].

4-1.2 23 AVENUE UNDERPASS

4-1.2.1 General

A. The LRT corridor requires a grade separation at the 23 Avenue and 111 Street NW intersection. The grade separation starts in the median of 111 Street south of Century Park LRT Station. The LRT track passes under 23 Avenue and the southbound lanes of 111 Street through a short tunnel. The LRT track exits the tunnel to the west of the southbound lane of 111 Street.

B. The proposed structure should be a 187 m twin box tunnel under the 23 Avenue and 111 Street intersection with a 50 m portal transition at the north end and a 90 m portal transition at the south.

C. The Design-Builder must design and construct the 23 Avenue Underpass in accordance with Section 3-3 [*Stormwater Management*] of this Schedule.

D. Several utilities conflict with the 23 Avenue Underpass. Relocation, design and construction must be in accordance with Section 3-5 [*LRT Corridor Utilities*] of this Schedule.

4-1.2.2 Geotechnical Considerations

A. A copy of the November 2022 Geotechnical Investigation – 23 Avenue Underpass is included as Disclosed Data. The Design-Builder is responsible for carrying out any additional investigations required to inform its Design and Construction. Baseline Geotechnical Conditions for the 23 Avenue Underpass indicating the conditions that are expected to be encountered are included in Appendix 4-1.2A [*23 Avenue Underpass – Geotechnical Baseline Conditions*].

B. Dewatering of the sand intertill / aquifer is required for construction of the underpass and the collected water must be disposed of away from the excavation/subgrade. The underpass structure Design must address the excess pore water pressure within the confined sand intertill / aquifer. A permanent dewatering system is not permitted therefore the effects of elevated/recovered pore water pressure must be addressed.

C. During the construction phase, a monitoring program of settlement and groundwater levels near the subject buildings and movements of the Geotechnical structures must be implemented. The monitoring program plan must be submitted to the City in accordance with Schedule 2 [*Submittal Review Procedure*] for Acceptance 90 days prior to implementation of the monitoring program. Subject buildings are defined as buildings located within 50 m of the excavation for the 23 Avenue Underpass. The Design-Builder must extend the groundwater and settlement monitoring beyond 50 m if significant settlements and/or changes in groundwater levels are observed. Geotechnical structures are defined as slopes, excavations, and/or retaining walls required for the Construction of the underpass.

D. Excavation cut slopes must be controlled based on soil conditions encountered, groundwater seepage, proximity of surcharge and stockpile loads and exposure.

E. Conventional subgrade preparation consisting of removal of organic and unsuitable soils and uniform compaction of subgrades and engineered fill to specified standards is required to provide subgrade support for the LRT tunnel and portals.

F. Portals must be designed and constructed as retaining walls.

4-1.2.3 Design

4-1.2.3.1 Tunnel and Portal Structures

A. The tunnel structure should consist of a reinforced concrete double box with walkways on either side of the interior walls with tunnel cross-connections. The thickness of walls and slabs forming the outer shell of the tunnel structure must be a minimum 600 mm unless justified by analysis.

B. Each tunnel box must contain six - 100 mm diameter conduit duct banks, providing a total of 12 conduit duct banks within the double box tunnel and portal structure.

C. Emergency egress from tunnel must be in accordance with NFPA 130.

D. Within the portal segments, OCS pole bases must be fixed to the base slab.

E. No structural elements are to penetrate the tunnel or portal base slab.

F. The tunnel roof slab must be either reinforced with stainless steel reinforcement or waterproofed with GCP Eliminator cold, liquid, spray-applied bridge deck waterproofing or approved equivalent.

G. Water ingress and corrosion protection must be in accordance with the HFDG.

H. Within the deeper portal segments horizontal struts should be used at the tops of tall retaining walls to optimize wall and base slab design and support the catenary system.

I. The Design Life for the structure must be 100 years.

J. Design must be in accordance with the HFDG except for load factors which are taken from the latest version of CAN/CSA S6.

K. The roof slab and supporting structure must be designed for CL-800 truck and lane loading as defined by CAN/CSA S6-19 and validated for City overload permit vehicles included in Section 4-3.3 [*Design Loads*].

L. LRV and work train loads are taken from the HFDG.

M. The crash criteria in the HFDG only applies to structural elements/walls in the underpass and not to the internal non-structural elements. Internal walls must be designed to be non-hazardous in a crash scenario but do not need to be designed to withstand an LRV derailment.

4-1.2.4 Construction Requirements

A. Access to retail and commercial properties east of 111 Street must be maintained during construction.

B. The 23 Avenue and 111 Street intersection must be detoured to accommodate construction of the tunnel and portals. Detour design and staging must be in accordance with Section 1-5 [*Transportation*

Management] of this Schedule. Detour staging must permit the relocation and construction of stormwater and utilities.

4-1.3 BLACKMUD CREEK LRT BRIDGE

4-1.3.1 Geometrics

4-1.3.1.1 General

A. The requirements of City of Edmonton D&CS must be met.

B. Impacts to the existing roadside and median barrier systems on 111 Street must be minimized. Refer to Section 1-5 [*Transportation Management*] of this Schedule.

4-1.3.1.2 Location

A. The Blackmud Creek LRT Bridge must be located west of the existing 111 Street roadway bridge. The minimum clear horizonal distance between any portion of the bridge superstructure or above ground substructure must not be less than 5.0 m.

4-1.3.1.3 Placement

A. The Blackmud Creek LRT Bridge must have a vertical variance of no more than 300 mm from the vertical profile of the 111 Street roadway bridge.

B. All components of the bridge must be located outside of the high water level contour for the 1:100 year flood event.

4-1.3.1.4 Egress

A. The superstructure must provide sufficient width for egress on both the east and the west side of the tracks as per the HFDG.

B. Egress must be provided along the entire length of the bridge that is confined by barriers.

4-1.3.2 Geotechnical

A. A geotechnical investigation has been completed and the report “Geotechnical Investigation – Blackmud Creek LRT Bridge” is included as Disclosed Data. The Design-Builder is responsible for carrying out any additional investigation required to inform its Design and Construction.

4-1.3.3 Piers

A. Pier shape and architectural finish must match the existing 111 Street roadway bridge.

B. Shallow footings on bedrock may be utilized for bridge piers constructed near the valley bottom, similar to the existing 111 Street roadway bridge, provided they are founded directly on competent bedrock, are well below the anticipated frost line, and there is no reasonable risk of loss of soil support adjacent to the pier footing.

4-1.3.4 Abutments

A. Abutments may be supported on pile foundations that do not retain earth in order to not add load to the existing ravine slopes.

   1. Abutment foundations are the structures between the main bridge spans over Blackmud Creek and the point where the transition slab is cast directly on ground material.

2. The transition slabs must be a minimum 6.0 m long and be cast-in-place or precast concrete.

3. The abutment foundations must be pile supported with a structural beam and roof slab and skirting walls or Omega fence a minimum of 1.8m height that enclose the area below the abutment roof over its entire length.

4. The void within the abutment foundations must be permanently enclosed to prevent trespass.

   a. Access must be provided for inspection purposes but must not be accessible by the public. This may be a single lockable hatch door accessed at each abutment backwall or wingwall with common keying for all hatch doors.

   b. The floor within the void must be graded and covered with either a concrete slope protection slab or rip-rap to provide a stable surface for abutment inspection and maintenance and to control potential erosion.

B. Conventional abutments consisting of headslopes and/or MSE walls may be considered.

1. If fill is placed on the existing valley slope (i.e., abutments situated adjacent to abutments of existing southbound 111th Street bridge); the addition of abutment fill (headslopes or MSE walls) may decrease stability of the existing valley slope and may potentially induce deep-seated slope instability. The proposed geometry of the abutment fill and surrounding valley slope must meet or exceed a minimum factor of safety of 1.5 against shallow and deep-seated potential failure mechanisms. The Design-Builder must submit a slope stability analysis confirming that the required factor of safety has been achieved in their Design, and that the added fill does not negatively impact the foundations of the existing 111th Street bridges.

2. Settlement at the abutment fill locations also needs to be considered and must comply with the requirements of Section 4-3.8 [*Geotechnical*]. Instrumentation must be utilized to monitor the settlement of abutment fills (settlement plates and/or horizontal shape accel arrays) to ensure that settlement targets are achieved.

3. Valley wall stability must be monitored utilizing slope inclinometers installed at locations satisfactory to the City to demonstrate that the proposed measures do not impact stability.

4-1.3.5 Drainage

A. The Design-Builder must design and construct drainage in accordance with Schedule 5, Part 3, Section 3-3 [*Stormwater Management*].

4-1.3.6 Durability

A. The SUP must have a HPC wearing surface. The HPC wearing surface must be a minimum of 50 mm thick and reinforced with steel fibres and/or stainless steel reinforcement (if the wearing surface is sufficiently thick to provide sufficient cover).

B. Sulphate resistant Portland cement must be used for all concrete in contact with ground or groundwater.

C. Concrete finishes must comply with the requirements of Section 4-4.6.23 [*Concrete Surface*] of this Schedule.

D. The soil is characterized as unusually corrosive. If steel piles are used, they must be designed to account for corrosion over the service life of the structure. Authenticated documentation in the form of a report or memo outlining the predicted corrosion rate and advising on required sacrificial steel thickness for the steel piles must be submitted to the City for review. The design code used to determine corrosion rate and sacrificial thickness must be indicated on the Design Drawings.

4-1.3.7 Sidewalk and Shared Use Path

A. A SUP must be constructed along the west side of the Blackmud Creek LRT Bridge with a minimum width of 4.2 m.

B. The SUP must have a minimum cross slope of 2%.

C. Locations of deck drains may be placed at the face of the barrier on the east side of the SUP, on both sides od the north abutment and on the south side of the south abutment. Deck drains may not under any circumstances discharge into the ravine or creek.

4-1.3.8 Loading

A. The SUP must be designed to support the following loads:

1. the maintenance vehicle load specified in CAN/CSA S6, Section 3.8.11;
2. pedestrian loads; and
3. the City Inspection Vehicle ASPEN A-40, details of which are included as Disclosed Data.

4-1.3.9 Deck

A. The deck may be full depth cast-in-place concrete or follow the requirements for Section 4-3.18.1.C.

B. The track must be connected to the concrete deck by direct fixation fasteners. Rail-structure analysis is required for this structure in accordance with the requirements of the HFDG Section 9.9.8 – "Rail-Structure Interaction Demands". An authenticated rail-structure interaction analysis report, independently checked by another Designer, must be submitted for review by the City.

4-1.4 111 STREET ROADWAY BRIDGE WIDENING

4-1.4.1 General

A. Modifications must be in accordance with this Schedule, at a minimum:

1. On the approaches:
    a. Removal of existing west bicycle railing and demolition of existing west traffic barrier.
    b. Demolition of west traffic barrier support beam and abandonment of associated H-pile.
    c. Removal of existing pedestrian railing and demolition of pathway slab and top of wingwall.
    d. Construction of new F-type west traffic barrier with bicycle railing mounted on wingwall.
    e. Extension of approach slab to face of new barrier and extension of pavement.
    f. Abandonment of existing duct bank.
2. On the deck:
    a. Removal of existing west bicycle railing and demolition of existing west traffic barrier.
    b. Removal of existing pedestrian railing and light poles and top portion of pathway deck including embedded utility ducts.
    c. Removal of west deck drains.

d. Construction of new F-type west traffic barrier with bicycle railing.

e. Extension of deck concrete to face of barrier.

f. Extension of pavement to face of barrier and construction of new deck drains and light poles.

B. The approach road, approach slab, roof slab, and bridge must be modified as required to remove the sidewalk and add a third driving lane to the existing 111 Street roadway bridge.

C. The new F-Type west traffic barrier must accommodate the new conduit.

4-1.4.2 Existing Information

A. Drawings for the existing 111 Street roadway bridge over Blackmud Creek are included as Disclosed Data.

4-1.4.3 Approach Slab

A. Work must include:

1. demolition and removal of the existing west traffic barrier, bicycle railing, and pedestrian railing;
2. demolition of the existing pathway slab and top of wingwall;
3. demolition of the existing traffic barrier support beam;
4. abandonment of the H-Pile previously supporting the traffic barrier support beam;
5. abandonment of the existing duct bank;
6. extending the roadway approach slab to the face of the new barrier;
7. placement of new pavement; and
8. new F-Type west traffic barrier with bicycle railing.

B. The extended roadway slab must follow the same constant cross slope of 2.5% and match the thickness of the existing roadway slab.

4-1.4.4 Roof Slab

A. Work must include:

1. partial depth removal of the existing roof slab, pathway slab, and top of wingwall. This includes the removal of the existing embedded utility ducts;
2. demolition of the existing west traffic barrier, bicycle railing, and pedestrian railing;
3. placement of a reinforced concrete topping, and new pavement; and
4. new F-Type west traffic barrier with bicycle railing.

B. The modified roof slab must follow the same constant cross slope of 2.5% and match the thickness of the existing roadway slab.

4-1.4.5 Deck

A. Work must include:

1. removal of existing deck drains and sealing the holes with grout;
2. partial removal of the top portion of the deck, including the embedded utility ducts;
3. demolition of the existing west traffic barrier, bicycle rail, pedestrian railing, and light poles;
4. placement of a reinforced concrete topping and new pavement; and
5. new F-Type traffic barrier with bicycle railing, and new light poles.

B. Accommodate new deck drain.

4-1.5 ANTHONY HENDAY DRIVE LRT BRIDGE

4-1.5.1 Geometrics

4-1.5.1.1 Location

A. The Anthony Henday Drive LRT Bridge is located south of Twin Brooks Station and crosses AHD.

4-1.5.1.2 Horizontal Alignment

A. The Anthony Henday Drive LRT Bridge must cross the AHD at a skew of 49°.

4-1.5.1.3 Vertical Clearance

A. Alberta Transportation and Economic Corridors requires a vertical clearance over the existing mainline and ramps of not less than 6.2 m, which includes a 0.1 m construction tolerance and 0.2 m allowance for future resurfacing. Minimum acceptable vertical clearance for Temporary Works is 6.0 m with a 0.1 m construction tolerance over any portion of the road surface which is not actively closed to vehicular traffic. A further reduction of vertical clearance for Temporary Works may be acceptable subject to approval by Alberta Transportation and Economic Corridors.

B. The north approach must maintain the minimum vertical clearance to the above powerlines, as specified by AltaLink in Section 4-1.5.3 [*AltaLink Lines*].

4-1.5.1.4 Substructure Placement

A. Piers and abutments must be placed to meet the horizontal clearance requirements of Section 4-3.7.2 [*Horizontal Clearances*] of this Schedule, for the current six lane condition. For future conditions of eight lanes, with an added lane on the inside and where the clear zone requirements cannot be met, the pier or abutment must be protected by a TL-4 barrier as required by the Road Safety Audit. If a TL-4 barrier is required, the pier or abutment must be set back a minimum of 3.0 m from the traffic face of the barrier.

4-1.5.1.5 Egress

A. The superstructure must provide sufficient width for emergency egress on the east side of the northbound track and west side of the southbound track as per the HFDG, with the exception that minimum clearance is measured from the static envelope instead of the dynamic train vehicle envelope.

4-1.5.2 Geotechnical

A. A geotechnical investigation has been completed and the report “Geotechnical Investigation – Anthony Henday LRT Bridge” is included as Disclosed Data. The Design-Builder is responsible for carrying out any additional investigation required to inform its Design and Construction.

4-1.5.3 AltaLink Lines

A. AltaLink has two vertical clearance requirements from overhead conductor at position of maximum sag:

   1. 10.4 m to top of rail and top of ground; and

   2. 4.3 m above line of sight of the OCS at support points on either side of the line.

B. Horizontal clearance of 9 m is required between the centreline of track or toe of slope, to the nearest transmission line support structure.

C. AltaLink will require a review of any objects more than 2 m in height, including OCS poles, within 20 m of transmission centerline.

4-1.5.4 Pipeline Right-of-Way

A. No embankment fill at the Anthony Henday Drive Bridge is permitted to encroach into the pipeline right-of-way that is east of the trackway. Road widening, SUP work, or non-embankment grading work may encroach into the adjacent pipeline right-of-way if permitted by the Pipeline Company.

4-1.5.5 Structure

A. The form of the bridge must comply with the design shown on the preliminary design drawings included in Appendix 4-1.5A [*Anthony Henday Drive LRT Bridge Preliminary Design Drawings*], with the exception of the options provided in Section 4-3.2.1 [*Anthony Henday Drive LRT Bridge*] of this Schedule.

B. The track must be connected to the concrete deck by direct fixation fasteners. Rail-structure analysis is required for this structure in accordance with the requirements of the HFDG Section 9.9.8 – "Rail-Structure Interaction Demands". An authenticated rail-structure interaction analysis report, independently checked by another Designer, must be submitted for review by the City.

C. An authenticated erection plan, including erection engineering drawings and calculations, must be submitted with sufficient time for approval of any necessary lane or road closures by the relevant authority.

4-1.5.6 Foundations

A. The geotechnical report indicates that shallow foundations are not considered practicable. The Design-Builder should use either driven steel piles or drilled cast-in-place concrete piles for all foundations.

4-1.5.7 Drainage

A. Deck run-off must not be discharged onto AHD. All deck run-off must be collected from the arch spans and drained at the approach span piers and/or abutments.

4-1.5.8 Durability

A. Sulphate resistant cement must be used for all concrete in contact with ground or groundwater.

B. The soil is characterized as "unusually corrosive". If steel piles are used, they must be designed to account for corrosion over the service life of the structure. Authenticated documentation in the form of a report or memo outlining the predicted corrosion rate and advising on required sacrificial steel thickness for the steel piles must be submitted to the City for review. The design code used to determine corrosion rate and sacrificial thickness must be indicated on the Drawings.

4-1.5.9 Lighting

A. The construction of the Anthony Henday Drive LRT Bridge requires the removal of one of the high mast lights in the median of AHD. The Design-Builder must evaluate lighting levels at this location in accordance with Alberta Transportation standards and add replacement lighting as required.

SECTION 4-2 – REFERENCE DOCUMENTS

A. Without limiting Section 4-2 [*Reference Documents*] of this Schedule and except as otherwise specified herein, Building Structures must comply with the NBCAE.

B. Without limiting Section 4-2 [*Reference Documents*] of this Schedule and except as otherwise specified herein, Transportation Structures must comply with:

   1. Canadian Highway Bridge Design Code, CAN/CSA S6-19 including errata:

      a. In CAN/CSA S6 references to the Regulatory Authority must be taken to be references to the City.

      b. The Canadian Highway Bridge Design Code Commentary, CAN/CSA S6.1 must not be a compliance document for this Agreement.

   2. City of Edmonton High Floor LRT Design Guidelines.

C. NBCAE and CAN/CSA S6 must be supplemented by other codes where required in this Part 4 [*Transportation Structures and Building Structures*].

D. The Design requirements in this Part 4 [*Transportation Structures and Building Structures*] may also be supplemented with additional Design requirements from other codes and standards not expressly listed in Section 4-2.E [*Reference Documents*] if permitted by the City, in its discretion. Prior to proceeding with using such additional Design requirements from such codes and standards, submit to the City all documentation or evidence requested by the City to demonstrate that the reliability index criteria of Section 4-3.1.1 [*Safety*] of this Schedule are met when the supplemental Design requirements are used. The City may require other modifications to the use of alternative codes and standards to comply with local practice and achieve a similar level of performance to a structure designed to S6.

E. Other codes and standards referenced in Section 4-3 [*Design Guidance/Requirements*] and Section 4-1 [*Description of Infrastructure*] of this Schedule are:

   1. Floor Vibrations Due to Human Activity, AISC Steel Design Guide 11 (AISC 11)

   2. CEB-FIP Model Code, Chapter 2, 1990 Edition (CEB-FIP)

   3. AASHTO, LRFD Bridge Design Specifications (AASHTO LRFD)

   4. PTI DC35.1, Recommendations for Prestressed Rock and Soil Anchors (PTI DC35.1)

   5. PTI M55.1, Specification for Grouting of Post-Tensioned Structures (PTI M55.1)

   6. EN 1991 - Part 2, Eurocode 1: Actions on Structures - Traffic Loads on Bridges (EC1)

   7. EN 1993 - Part 2, Eurocode 3: Design of Steel Structures - Steel Bridges (EC3)

   8. AASHTO, LRFD Bridge Construction Specifications (AASHTO LRFD BCS)

   9. AASHTO. LRFD Guide Specifications for Bridges Carrying Light Rail Transit Loads, (AASHTO LRFD LRT)

   10. AASHTO, Standard Specifications for Highway Bridges (AASHTO SSHB)

   11. AWS D1.5 Bridge Welding Code (AWS D1.5)

   12. PTI/ASBI M50.3 Guide Specification for Grouted Post-Tensioning (PTI/ABSI M50.3)

13. FHWA-NHI-05-039, Micropile Design and Construction Reference Manual
14. FHWA-NHI-14-007, Soil Nail Walls Reference Manual
15. FHWA-NHI-10-025, Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes
16. Fib Bulletin No. 34, Model Code for Service Life Design (FIB 34)
17. NFPA 130, Standard for Fixed Guideway Transit and Passenger Rail System (NFPA 130)
18. TAC Geometric Design Guide for Canadian Roads (TAC)
19. Alberta Transportation Standard Specification for Bridge Construction Edition 17 2020 (SSBC)
20. Alberta Transportation Products List February 2022
21. CAN/CSA A23.1/23.2, Concrete Materials and Method of Concrete Construction
22. CAN/CSA A23.3, Design of Concrete Structures
23. CAN/CSA A23.4, Precast Concrete Materials and Construction
24. CAN/CSA S269.1 Formwork and Falsework
25. PTI M50.1 Acceptance Standards for Post-Tensioning Systems
26. OPSS 1213, Material Specification for Hot Applied Rubberized Asphalt Waterproofing Membrane
27. OPSS 1215, Material Specification for Protection Board
28. CAN/CSA O86, Engineering Design in Wood

SECTION 4-3 – DESIGN GUIDANCE/REQUIREMENTS

4-3.1 GENERAL STRUCTURAL PERFORMANCE REQUIREMENTS

4-3.1.1 Safety

A. Building Structures must be designed to have a minimum reliability index in accordance with the requirements NBCAE.

B. Transportation Structures must be designed to have a minimum reliability index that complies with Section 3.5.1 of CAN/CSA S6.

C. Structures must meet the requirements for safety and security in the HFDG in terms of the accommodation of all intended uses, including all activities, in accordance with the High Floor Operations and Maintenance Parameters, and throughout their Design Life.

   1. Bridges and Stations must meet the life safety requirements of NFPA 130 and NBCAE, as applicable.

D. The Construction must not impact the structural integrity or safety of other structures.

E. In accordance with Clause 1.4.2.5 of S6, special consideration must be given to single load path structures and primary load carrying components with non-inspectable webs of Transportation Structures. These structures and elements must be designed to provide ductile behavior in the ultimate limit state and provide warning of failure prior to collapse.

4-3.1.2 Functionality

A. The geometry and details of the Structures must permit the safe operation, inspection and maintenance of the Infrastructure, including requirements during emergencies as described in the High Floor Operations and Maintenance Parameters.

4-3.1.3 Serviceability/Durability

A. There must be no noticeable or measurable deterioration of the performance or ability of a Structure to carry load and no deterioration detrimental to the appearance of a Structure over its Design Life.

B. In accordance with the HFDG Section 9.1.2 – “Service Life and Design Life”, non-replaceable components must have a service life to match at least the design life of the entire structure. Replaceable components must have service life consistent with the schedule of maintenance and replacement specified during design.

C. Connections and interfaces between structural components must be designed and detailed to accommodate any tolerances and deviations that could reasonably be expected to occur between the design and fabricated dimensions and elevations of the components being connected. The connections must accommodate the tolerances and deviations without field modification, and in such a way that unanticipated stresses are not introduced into the structural components.

4-3.2 SUSTAINABLE URBAN INTEGRATION (SUI)

A. Part 2 [*Sustainable Urban Integration and Landscape Architecture*] provides the overarching requirements for SUI and the specifications for Landscape Architecture. SUI goals and objectives outlined in Schedule 5, Part 2 Section 2-1.3 [*SUI Goals and Objectives*] must be met. Additional specific SUI requirements have been incorporated in this Part.

4-3.2.1 Anthony Henday Drive LRT Bridge

A. Design Context

1. This Section 4-3.2.1 [*Anthony Henday Drive LRT Bridge*] sets out the SUI requirements for the Anthony Henday Drive LRT Bridge.

2. The location of the Anthony Henday Drive LRT Bridge over a heavily trafficked highway in an area of generally flat terrain makes it highly visible to motorists using the highway, as well as the residential neighbourhood south of the highway.

B. General Requirements

1. The form of the Anthony Henday Drive LRT Bridge must comply with the preliminary design drawings included in Appendix 4-1.5A [*Anthony Henday Drive LRT Bridge Preliminary Design Drawings*]. Form, in this context, relates to the span-rise-ratio, support locations and overall dimensions. Minor changes to dimensions and section sizes required to meet Project Requirements or for the purposes of structural efficiency are permitted. The bridge must comprise two steel network arches with concrete deck main spans over AHD and two concrete girder approach spans.

2. The piers and abutments of the Anthony Henday Drive LRT Bridge must be constructed from reinforced concrete. The colour of the pier and abutment concrete must match the colour of the superstructure concrete.

C. Pier Requirements

1. Piers must be constructed at the locations shown on the preliminary design drawings included in Appendix 4-1.5A [*Anthony Henday Drive LRT Bridge Preliminary Design Drawings*].

2. The center pier in the median of AHD must comply with the design shown in Appendix 4-1.5A [*Anthony Henday Drive LRT Bridge Preliminary Design Drawings*].

    a. The pier shaft must be rectangular in cross section with chamfered corners and must be finished with architectural details similar to the design shown on the preliminary design drawings included in Appendix 4-1.5A [*Anthony Henday Drive LRT Bridge Preliminary Design Drawings*].

    b. The bottom corners of the hammerhead pier cap must be chamfered in accordance with the dimensions shown on the preliminary design drawings included in Appendix 4-1.5A [*Anthony Henday Drive LRT Bridge Preliminary Design Drawings*].

    c. The pile cap must be buried a minimum of 300 mm below grade.

3. The piers between the main spans and approach spans may conform to the two-column design shown on the preliminary design drawings included in Appendix 4-1.5A [*Anthony Henday Drive LRT Bridge Preliminary Design Drawings*], or the single column hammerhead pier following the design of the center pier in the median of AHD.

4. For the two-column pier design:

   a. the column shaft diameter must not be greater than the dimension shown on the preliminary design drawings included in Appendix 4-1.5A [*Anthony Henday Drive LRT Bridge Preliminary Design Drawings*]; and

   b. a concrete pile cap to support the two concrete columns will be permitted in lieu of extending concrete drilled piles to form the columns above grade. If used, the pile cap must be buried a minimum of 300 mm below grade.

5. Drainage pipes must be concealed within the pier concrete.

D. Superstructure Requirements

1. The network arch spans must have a slender appearance with clean, uninterrupted lines.

   a. The arch ribs must be constructed from structural steel and shop painted. The colour of the topcoat must be submitted to the City for review.

   b. Field splices in the arch ribs using bolted connections must be minimized. Arch rib bracing may use cross bracing or Vierendeel truss arrangement. Only one arrangement of arch rib bracing must be used for the two main spans.

   c. The deck carrying the LRT tracks must be constructed from post-tensioned concrete. The cross section must comply with the dimensions shown on the preliminary design drawings included in Appendix 4-1.5A [*Anthony Henday Drive LRT Bridge Preliminary Design Drawings*]. The soffit of the deck over AHD must have a smooth, uninterrupted finish. Subject to review and acceptance by the City and Alberta Transportation and Economic Corridors, alternative materials and construction methods may be acceptable for the deck, provided that equivalence of aesthetics, durability, maintainability, geometry and clearances are demonstrated and that the materials and construction methods are suitable for use in the TUC. At a minimum, suitability of materials and construction methods must be demonstrated by compliance with Alberta Transportation and Economic Corridors standards and specifications.

   d. The deck must be suspended from the arch ribs by inclined hangers. The number and inclination of the hangers must be similar to the design shown on the preliminary design drawings included in Appendix 4-1.5A [*Anthony Henday Drive LRT Bridge Preliminary Design Drawings*]. The colour of the hanger sheathing must match the colour of the paint used for the network arches as much as practicable.

2. The approach spans must comprise precast concrete girders supporting the concrete deck, which carries the LRT tracks. Voided abutments in lieu of bridge spans will not be permitted.

E. Deck drain pipes must not be visible when the bridge is seen from the side. Deck run-off from the main spans must be directed away from AHD to be discharged.

4-3.2.2 23 Avenue Underpass

A. Design Context

1. This Section 4-3.2.2 [*23 Avenue Underpass*] sets out the SUI requirements for the 23 Avenue Underpass.

2. The north and south portals of the 23 Avenue Underpass will be the only elements of the structure visible. The portals are located within the roadway boulevards and medians, thus the SUI features in the underpass will be visible to LRT passengers.

B. General Requirements

1. The form of the 23 Avenue Underpass should contain walls with consistent appearance within material types, with or without a central wall between the tracks.

2. The approach structures and portals must either be constructed in cast-in-place concrete or an alternative non-combustible material. If an alternative non-combustible material is used to construct the walls, it:

   a. must provide the required insulation to the secant pile walls;

   b. must have a maintenance free design life of minimum 25 years;

   c. must accommodate drainage of the secant pile walls;

   d. must be detailed to be replaceable; and

   e. must be detailed to prevent damage to the material resulting from the conditions of installation (e.g. so that the material is not subject to damage from freeze-thaw cycles).

3. Additional SUI considerations for 23 Avenue Underpass include:

   a. the safety railing must be designed as a “guard” in compliance with the requirements of NBCAE;

   b. safety fencing on the tops of portals must be screened and integrated with landscaping and plantings;

   c. construction joints must be carefully integrated into the architectural theme;

   d. lighting must consider SUI, as well as LRT and the 23 Avenue and 111 Street roadway intersection operational requirements; and

   e. architectural treatments must incorporate anti-graffiti measures as per the HFDG.

4-3.3 DESIGN LOADS

4-3.3.1 General

A. This Section 4-3.3 [*Design Loads*] sets out loads to be accounted for in the design of Structures, which are additional to those in CAN/CSA S6 and the NBCAE, as applicable.

B. All design loads must satisfy the requirements in the HFDG Section 9.9 – “Design Loads”, with the following amendments:

1. On-track Vehicle Vertical Live Load Model

2. Light Rail Vehicle Wind Load

3. The City Inspection Vehicle

4. Seismic Loads

4-3.3.2 On-track Vehicle Vertical Live Load Model

A. On-track vehicle vertical live loads must be used as live loads, ***"L"***, in CAN/CSA S6 and NBCAE.

B. The minimum on-track vertical live load must be the load presented in the HFDG Section 9.9.3 – "Live Load – Design Vehicles". Both the Light Rail Vehicle (LRV) (HFDG Figure 9.3) and Work Train (HFDG Figure 9.4) must be considered.

4-3.3.3 LRV Wind Load

A. The wind load on LRVs must be used as a wind load "V" in CAN/CSA S6.

B. The horizontal drag coefficient on the LRVs must be a minimum of $C_h = 2.0$.

4-3.3.4 City of Edmonton Inspection Vehicle

A. The City Inspection Vehicle vertical live load, defined in Section 4-1.3.8 [*Loading*], must be used as live load, "***L***" in CAN/CSA S6. It is not required to include DLA and braking forces associated with this Vehicle.

4-3.3.5 Seismic Loads

A. Transportation Structures must have a CAN/CSA S6 seismic importance category of "Other Bridges".

B. Building Structures must have an NBCAE seismic importance category of "Normal".

4-3.3.6 Stations Live Load

A. Publicly accessible areas of Stations and all Platforms must be designed for a minimum live load of 4.8 kPa.

4-3.3.7 Anthony Henday Drive LRT Bridge Over Height Collision

A. The Anthony Henday LRT Bridge must be designed for the prevention of collapse due to over height vehicle collision loads from vehicles on the underpassing roadways. The collision loads must be applied to the tie beam at any location over the underpassing roadway (including within the shoulders of the underpassing roadway) to produce the maximum loading effects. The collision load must generally be applied to the bottom outside corner of the concrete tie beam (or bottom flange if a girder is used), although consideration must be given to the possible effects of the collision load being applied to the outer vertical plate of the tie beam if a girder is used. The Designer must provide a suitable load path to transfer the load to the bearings, supports and foundations. Each structure element in the load path must be considered with the loads provided.

1. In addition to the load combinations provided in the HFDG, the Anthony Henday Drive LRT Bridge must be designed for:

   $\alpha DxD + \alpha DxE + \alpha DxP + 0.75xL + H$ ULS Combination 12

   a. For this combination, both tracks must be assumed to be in service

   b. The following collision loads must be considered as collision loads "H" under ULS Combination 12 for all elements of the bridge structure.

      i. A static force of 1250 kN applied to the tie beam in a direction parallel to the underpassing roadway.

      ii. A static force of 625 kN applied to the tie beam in a direction normal to the underpassing roadway.

iii. The load parallel to the underpassing roadway must be considered to act separately to the load normal to the underpassing roadway. The loads must be considered to act as point loads on the tie beam in any direction between the horizontal and vertical.

2. The bridge bearings must also be designed with an additional equivalent static live load force of 625 kN, applied to the bridge in a direction parallel to the underpassing roadway and considered under SLS Combination 1 or an alternative means of connection provided between the superstructure and substructure provided to take this force at each bearing location.

B. Designers must address system redundancy by considering what the capacity of the tie beam might be after an over height collision event. The Designer must assess the potential for damage due to the over height vehicle collision and must use good common engineering judgement in the assessing how much of the capacity could be compromised. At a minimum, either a reduction in sectional area of the impacted tie beam of 25% at the collision location or loss of the entire bottom flange of the girder, whichever results in the loss of the greater sectional area, must be accounted for. This should include severing of any reinforcement or prestressing steel in the affected zone of a concrete tie beam. Damage scenarios for other structural arrangement(s) must also be evaluated. The Designer must prove sufficient immediate post-collision capacity exists at the Ultimate Limit States (ULS Combinations 1 and 2) to prevent collapse of the structure in both the minimum damage scenario defined above and the any damage scenarios for other structural arrangements evaluated by the Designer. This verification of post-collision capacity may include the contribution of system redundancy and alternative load pats. The on-track live loads used for these immediate post-collision checks may be reduced to 75% of the Light Rail Vehicle on one track. The live load factor applied may be reduced to 1.05.

C. Designers must address element redundancy by considering the following damage scenarios at any location above the paved roadway surface of Anthony Henday Drive and ensuring that the tie beam element has sufficient redundancy to avoid collapse of the structure without relying on system redundancy. These damage scenarios should be considered at the Ultimate Limit States (ULS Combinations 1 and 2). The on-track live loads used may be reduced to 75% of the Light Rail Vehicle on one track. The live load factor applied may be reduced to 1.05.

1. Punching failure through the web of a steel or concrete box girder at the point of impact, this can be modelled as total loss of the web plate over a distance of 0.5m.

2. Local distortion or failure of web of steel girders caused by application of collision load to bottom flange. This damage scenario is dependent on the section properties of the girder and must be modelled by the Designer and included in the calculation package.

3. Local damage to the bottom flange of a steel girder including a crack through 50% of the sectional area.

4. Local damage to the bottom flange of a concrete girder including severing of prestressing strands in the bottom 200 mm of concrete.

5. Designers must show recommended repairs for the listed damage scenarios on the drawings and ensure that the structure can carry full design loading with no reduction in reliability index or load factors if these repairs are implemented (for example steel girders must be designed for loss of section resulting from the installation of splices required for repair).

4-3.4 LIMIT STATES DESIGN

A. Where the design of the structural elements of a Structure is affected by a combination of live loads for Building Structures and for Transportation Structures, limit states design must be in accordance with the requirements for Transportation Structures.

B. Notwithstanding Section 4-3.4A [*Limit States Design*], limit states design of the Llew Lawrence OMF must be in accordance with the requirements of Building Structures for all loads, including Light Rail Vehicle live loads.

C. Transportation Structures must satisfy the requirements in the HFDG Section 9.10 – "Limit States Design", with the following amendments:

   1. In addition to the load combinations defined in the HFDG Section 9.10.1 – "Ultimate Limit State", Transportation Structures erected using segmental concrete construction must satisfy the load combination requirements in Section 5.12.5.3.4 "Construction Load Combinations at Strength Limit States" of AASHTO LRFD.

4-3.5 MATERIALS

A. This Section 4-3.5 [*Materials*] sets out minimum requirements for materials permitted for use in Structures.

4-3.5.1 Concrete

A. Concrete classes must conform to Table 4-3.5.1-1 [*Concrete Classes*].

B. All concrete must be normal weight concrete.

C. Unless otherwise specified, concrete in Transportation Structures, must as a minimum, meet the requirements for Class C concrete, except that piles must as a minimum meet the requirements for Pile Concrete.

D. Unless otherwise specified, concrete in Building Structures must, as a minimum, meet the requirements for Class B Concrete.

E. All concrete classes except Class Pile Concrete Tremie and Class SCC must have a slump no greater than 200 mm, except that Class Pile Concrete: General may have a slump no greater than 230 mm. Slumps higher than 100 mm must be obtained using superplasticizers.

F. Class SCC concrete must have a maximum spread diameter of 800 mm.

G. The size of coarse aggregate and air content must conform to CSA A23.1.

H. The maximum practicable size of coarse aggregate must be used.

I. The fly ash content for concrete not containing silica fume must not exceed 30% by mass of cementing materials unless otherwise specified in Section 4-3.6 [*Durability*] of this Schedule.

J. For Building Structures, where the concrete is not exposed to freeze thaw, sulphates or chlorides, air entrainment is not required.

K. Dry cast concrete is not permitted.

L. Class Pile Concrete: General with a slump higher than 200 mm must be trial batch tested to demonstrate workability retention, including slump flow, visual stability index, and air content, at 30 minutes, 60 minutes and 90 minutes after batching.

M. Class Pile Concrete: Tremie must have a slump flow between 400 mm and 550 mm and must be trial batch tested:

   1. to demonstrate workability retention, including slump flow, visual stability index, and air content, at 30 minutes, 60 minutes and 90 minutes after batching;

2. to confirm that a Visual Stability Index Rating of 0 and a Hardened Visual Stability Index Rating of 1 or less determined in accordance with AASHTO R81 Standard Practice for Static Segregation of Hardened Self-Consolidating Concrete Cylinders are achieved;

3. to demonstrate timing for initial and final setting;

4. to demonstrate compressive strengths at 3, 7 and 28 days are in accordance with the Project Requirements; and

5. to meet the requirements of Section 4-4.6.4.4 [*Trial Batches*] of this Schedule for concrete mixes containing hydration stabilizing admixtures.

Table 4-3.5.1-1 Concrete Classes

Class of Concrete[3]	Minimum Specified Compressive Strength at 28 Days (MPa)	Range of Slump (mm)	Max. Water/ Cementing Materials Ratio
B	25	100 ± 30	0.45
C	35	100 ± 30	0.40
HPC [1]	45	120 ± 30	0.38
Pile	30 [2]	130 ± 30	0.42
Tremie	30	N/A	
SCC	35	N/A	0.40

Notes:

1. *Class HPC concrete must contain silica fume. Additional requirements for concrete containing silica fume are set out in Section 4-3.5.2* [*Additional Requirements for Concrete Containing Silica Fume*] *of this Schedule.*
2. *Pile Design strength must not be greater than 30 MPa.*
3. *These classes do not directly align with the Alberta Transportation Economic Corridors Design and Construction Standards.*

4-3.5.2 Additional Requirements for Concrete Containing Silica Fume

A. The concrete mix must include silica fume and fly ash as supplementary cementing materials in combination with compatible air entraining, water reducing and/or super plasticizing admixtures, as required, by the mix design.

B. The gradation limits for the fine aggregate must conform to CAN/CSA A23.1, except that the amount of material finer than 160 μm must not exceed 5%.

C. Coarse aggregate must conform to CAN/CSA A23.1 and the maximum combination of flat and elongated particles (4:1 ratio), as determined by CAN/CSA A23.2-13A (Procedure A), must not exceed 10% of the mass of coarse aggregate.

D. Minimum Type GU cement content (excluding supplementary cementing materials) must be 335 kg/m³. Type HS cement must not be used.

E. Sum of silica fume and fly ash by mass of cementing materials must be 17% to 20%.

F. Silica fume by mass of cementing materials must be 6% to 8%.

G. Fly ash by mass of cementing materials must be 11% to 15%.

H. Resistance to chloride ion penetration must be determined in accordance with ASTM C1202 on duplicate laboratory moist cured samples at 28 days. The average of all tests must not exceed 1000 coulombs, with no single test greater than 1250 coulombs. When only two test values are used to calculate the average coulomb rating, no test must exceed 1000 coulombs. For Class HPC concrete with steel fibres, testing must be done without the presence of the steel fibres.

I. An air void spacing factor must be determined in accordance with ASTM C457 modified point-count method at 100 times magnification. The average of all tests must not exceed 230 μm with no single test greater than 260 μm. When only two test values are used to calculate the average air void spacing, no test must exceed 230 μm.

4-3.5.3 Concrete Reinforcement

A. Concrete reinforcement must be carbon steel or stainless steel and must conform to the following standards.

B. Carbon steel reinforcing steel must conform to CAN/CSA G30.18M, with a minimum yield strength of 400 MPa.

C. Stainless steel reinforcing steel must conform to ASTM A276 and ASTM A955M (including Annex 1.2 or 1.3) with a minimum yield strength of 420 MPa. Austenitic grades must meet the requirements of ASTM A262, Practice E. Duplex grades must meet the requirements of ASTM 1084, Method C by demonstrating no presence of detrimental phases. The UNS designations must be UNS S31653, S31803, or S32304.

4-3.5.4 Type 1c Concrete Sealer

A. Type 1c concrete sealer must be selected from the Alberta Transportation Products List.

4-3.5.5 Deformed Welded Wire Mesh

A. Deformed welded wire mesh must conform to ASTM A1064, Grade 70 (f_y = 485 MPa) with a minimum yield strength based on the 0.2% offset method. Welded wire mesh reinforcement must be able to attain a minimum elongation of 4% at ultimate strength. Testing for elongation must be in accordance with the Tension Test specified in ASTM A1064 with the following modifications:

   1. The minimum test gauge length must be 100 mm;
   2. 100% of the tests must be across the welds; and
   3. The extensometer must not be removed until 4% elongation has been attained.

4-3.5.6 Prestressing Steel

A. Prestressing steel strand must conform to ASTM A416 for low relaxation strand.

B. Prestressing steel bars must conform to CAN/CSA G279.

4-3.5.7 Structural Steel

4-3.5.7.1 Transportation Structures

A. The structural steel for Transportation Structures must conform to CAN/CSA G40.20/G40.21 and the following requirements:

   1. Primary load carrying members including girders, arches, splice plates and all materials welded to primary load carrying members: Grade 350AT CAT 3.

2. Ungalvanized bearing and bracing materials bolted to primary load carrying members: Grade 350A.

B. Grade 350WT CAT 3 and Grade 350W may be substituted for Grade 350AT and Grade 350A correspondingly where the structure is galvanized.

4-3.5.7.2 Building Structures

A. Structural steel for Building Structures must conform to the following requirements:

1. steel for open web steel joists must comply with CAN/CSA G40.20/G40.21, Type W with a minimum yield strength of 260 MPa for chord sections and web material;

2. W steel shapes must comply with CAN/CSA G40.20-13/G40.21, Grade 350W or ASTM A992/A992M, Grade 50;

3. HSS steel shapes must comply with CAN/CSA G40.20/G40.21, Grade 350W, Class C;

4. channel and angle steel shapes must comply with CAN/CSA G40.20-13/G40.21, Grade 350W;

5. other steel shapes and plates must comply with CAN/CSA G40.21, Grade 300W; and

6. cold-formed structural steel must conform to the material standards in CAN/CSA S136 and comply with ASTM A653/A653M Grade 230.

B. Galvanizing for steel shapes must comply with ASTM A123/A123M, or CAN/CSA G164 for irregularly shaped articles, and must have a minimum 600 g/m^2 coating.

C. Metal deck must be galvanized sheet steel conforming to ASTM A653, Grade 230 with a zinc coating that protects the steel for the Design Life of the Structure.

D. Roof and floor decks must conform to CSSBI 10M and 12M, respectively, and must be designed to act as diaphragms.

4-3.5.7.3 Structural Bolts and Anchor Rods

A. Structural bolts must conform to ASTM F3125 Grade A325/A325M heavy hex style except that bolts in contact with CAN/CSA G40.20/G40.21 Grade 350 AT or 350A steel must conform to ASTM F3125 Grade A325, Type 3.

B. Nuts must be heavy hex style and must conform to ASTM A563/A563M.

C. Hardened washers must conform to ASTM F436/F436M.

D. Anchor rods must conform to ASTM F1554 (Grade 36 or Grade 55 Weldable) or ASTM A193 Grade B7, unless specified otherwise for the typical roadside accessory structure in question (traffic signal, streetlight etc.) in City Design & Construction Standards.

E. Nuts and washers must be of equal or greater strength than the bolts or anchor rods to which they attach.

F. Bolts, nuts and washers must be finished to match the members to which they attach.

4-3.5.8 Ground Anchors

A. Ground anchors must be prestressed steel bars and must conform to the requirements of PTI DC35.1.

B. The use of prestressed steel strands is not permitted.

C. Anchorages of ground anchors must comply with the requirements of PTI DC35.1.

4-3.5.9 Micropiles

A. Permanent steel casing/pipe (if required) must:

1. come in one section without splices or joints; and

2. meet the tensile requirements of ASTM A252, Grade 3, except the yield strength must be a minimum of 345 MPa to 552 MPa.

B. Steel bars must be deformed bars with a yield strength of 520 MPa in accordance with ASTM A615.

4-3.5.10 Soil Nails

A. Soil nail tendons must be continuous, solid steel bars without splices or welds. Soil nail tendons must be Grade 420 or 520 steel and must meet the requirements of ASTM A615.

4-3.5.11 Structural Wood

A. The use of structural wood in Structures is only permitted in Building Structures.

B. Structural wood must conform to the material standards in CAN/CSA O86, Engineering Design in Wood.

C. Structural wood must be installed in such a manner that moisture is not trapped within the wood.

D. Structural wood must have the preservatives and treatments to provide protection from exposure in accordance with CAN/CSA O80 Series "Wood Preservation".

4-3.5.12 Masonry

A. The use of masonry material in Structures is only permitted in Building Structures, and for Transportation Structure veneers.

B. Concrete block masonry must conform to CAN/CSA S304.1.

C. Masonry must conform to CAN/CSA A371.

D. Veneers used in Transportation Structures must not be considered a protection system against chloride attack.

4-3.5.13 Grout

A. Unless otherwise specified, grout in Transportation Structures must be a non-shrink grout on the Alberta Transportation Products List.

B. Unless otherwise specified, grout in Building Structures must be a non-shrink grout.

4-3.6 DURABILITY

A. This Section 4-3.6 [*Durability*] sets out minimum durability requirements for structural components of Building structures and Transportation Structures unless otherwise specified.

B. The minimum durability requirements for Building Structures are specified in CAN/CSA S478, *Guideline on Durability in Buildings*.

C. The minimum durability requirements for Transportation Structures are specified in CSA-S6 Section 2.

D. Structures must have sufficient durability to meet the Design Life requirements.

E. The Stray Current levels defined in the Stray Current Mitigation Program must be accounted for when determining the Design Life of a Structure.

4-3.6.1 Splash Zone Surfaces

A. All surfaces within a Splash Zone Surface that are exposed to de-icing chemicals, freeze thaw cycles, and prolonged moisture conditions must be designed and detailed in accordance with CAN/CSA S6. If a portion of a structural element is within the Splash Zone Surface, then the entire structural element must be considered to be within the Splash Zone Surface.

B. Prepare and submit a report to the City justifying the selection of materials to be used in all concrete within 300 mm of any Splash Zone Surface, so as to achieve the minimum Design Life of the Structure.

C. Concrete reinforcing in Splash Zone Surfaces for Building Structures and Transportation Structures must be stainless steel in accordance with the requirements in Section 4-3.5.3 [*Concrete Reinforcement*] of this Schedule unless noted otherwise.

D. Trackway slabs and Building Structure surfaces may be exempted from using stainless steel reinforcement by the City, if a report is prepared and submitted to the City justifying the selection of materials to be used in all concrete within 300 mm of any Splash Zone Surface, to achieve the minimum Design Life of the Structure.

E. The analysis justifying the selection of materials to be used must be a full probabilistic verification in accordance with fib-34, "*Model code for Service Life Design*", using test data of the applicable concrete mix and the probabilistic distributions and values proposed in fib-34, with the exception of the following parameters in Table 4-3.6.1-1 [*fib Bulletin-34 Full Probabilistic Method Parameters*] or by an alternate method Accepted by the City.

Table 4-3.6.1-1 fib Bulletin-34 Full Probabilistic Method Parameters

Parameter	Variable	Value
Initial Chloride Content, C_0 (wt.-%/c)	μ	0.055%
	σ	0.028%
Critical Chloride Content, C_{crit} (wt.-%/c)	μ	0.20%
For Carbon Steel	σ	0.08%
	a	0.1%
	b	0.6%
Chloride content at depth Δx, $C_{s,\Delta x}$ (wt.-%/c)		
Exposure Class XD4	μ	4%
	σ	3%

Parameter	Variable	Value
Exposure Class XD3	μ	3%
	σ	2.25%
Exposure Class XD2	μ	2%
	σ	1.5%
Exposure Class XD1	μ	1%
	σ	0.75%

F. Where the Exposure Classes are defined as follows:

1. **Exposure Class XD4** means any surfaces with direct application of de-icing salts, including roadway, Platform, Heritage Valley North Station passenger loading area, SUP and sidewalk surfaces, surfaces within 30 m (measure horizontally) of a roadway crossing or 2 m of a pedestrian crossing (measured horizontally), and interior surfaces of Building Structures within 2 m of an entrance adjacent to a surface exposed to de-icing salts.

2. **Exposure Class XD3** means any surfaces within less than 1.5 m (measured horizontally) of a surface with direct application of de-icing salts or surfaces buried less than 300 mm below a surface with direct application of de-icing salts.

3. **Exposure Class XD2** means any surfaces within more than 1.5 m, but no more than 3 m (measured horizontally), of a surface with direct application of de-icing salts, surfaces buried more than 300 mm, but less than 1 m, below a surface with direct application of de-icing salts, or surfaces buried less than 300 mm below surfaces within 1.5 m of a surface (measured horizontally) with direct application of de-icing salts; or within 10 m (measure vertically) of a surface with direct application of de-icing salts, the airborne spray zone.

4. **Exposure Class XD1** means any surfaces exposed to splash further than 3 m (measured horizontally) from a surface with direct application of de-icing salts, or surfaces buried more than 300 mm below surfaces within 1.5 m of a surface with direct application of de-icing salts.

G. The report must define testing standards, procedures, and testing frequency during Construction to validate the assumptions used in the report.

H. CAN/CSA G40.20/G40.21, Grade 350A and 350AT steel must not be used within a Splash Zone Surface unless galvanized in accordance with Section 4-4.11.8.2 [*Galvanizing*] of this Schedule.

I. The fly ash content in concrete used as a Splash Zone Surface must not exceed 25% and the cement content of the concrete must be greater than 300 kg/m^3.

J. Concrete within splash zones must be resistant to chloride ion penetration according to CSA A23.2-23C.

K. All Building Structures steel within a Splash Zone Surface, excluding benches, leaning rails, waste receptacles, and waste and recycling receptacles, must be galvanized or be stainless steel.

L. Metal components of benches, leaning rail, waste receptacles, and waste and recycling receptacles must be aluminum, galvanized, or stainless steel.

4-3.6.2 Clear Concrete Cover

A. The nominal concrete covers for Transportation Structures specified in Table 8.5 of CAN/CSA S6 must be considered as minimum concrete covers with no negative tolerance on the nominal cover.

B. For MSE wall panels, minimum cover to concrete reinforcement must be 50 mm (excluding any additional thickness required for aesthetic surface treatment) on both front and back faces of the panels, and concrete reinforcement must be electrically isolated from soil reinforcement attachment hardware.

4-3.6.3 Concrete Surfaces

A. Concrete placement procedures must be prepared for each concrete mix design to outline the procedures for concrete placement to minimize the probability of the concrete deficiencies noted in Section 4-4.6.22 [*Concrete Deficiencies*] of this Schedule.

4-3.6.4 Steel Surfaces

4-3.6.4.1 General

A. All steel surfaces, except CAN/CSA G40.20/G40.21, Grade 350A and 350AT steel surfaces, must be protected by hot-dip galvanizing or by metallizing.

B. Galvanizing must conform to ASTM A123/A123M and A385/A385M.

C. Metallizing must be a minimum of 180 microns thick and must conform to AWS C2.23M, “Specification for the application of thermal spray coatings (metallizing) of aluminum, zinc, and their alloys and composites for the corrosion protection of steel”.

D. Paint based coating systems are not permitted unless otherwise specified.

4-3.6.4.2 Permanent Steel Piles

A. Steel piles that are exposed above ground must be hot-dip galvanized to 1.0 m below ground.

B. Steel piles must be designed to account for long-term section loss by providing additional sacrificial area, by hot-dip galvanizing or by a combination of both. If sacrificial area is used the Design-Builder must submit calculations demonstrating the adequacy of the additional area used.

4-3.6.4.3 Voids within Steelwork

A. All voids must be inaccessible to birds and other wildlife, with gaps and holes no larger than 20 mm.

4-3.6.4.4 Ground Anchor Corrosion Protection

A. All permanent ground anchors, including anchorages and couplers, must be provided with a Class 1 or “double corrosion” protection system in accordance with the requirements of PTI DC35.1.

4-3.6.4.5 Micropile Corrosion Protection

A. Micropile steel bars must be encased in grout-filled, corrugated HDPE sheathing (encapsulation) for double corrosion protection in accordance with the requirements of FHWA-NHI-05-039.

4-3.6.4.6 Soil Nail Corrosion Protection

A. All permanent soil nails must be provided with a Class A Corrosion Protection (encapsulation) in accordance with the requirements of FHWA-NHI-14-007.

4-3.6.4.7 Post-Tensioning Tendon Corrosion Protection

A. All post-tensioning tendons must be provided with a “Protection Level 2” protection system in accordance with the requirements of PTI/ASBI M50.3.

B. Post-tensioning tendons must not be located within 300 mm of a Splash Zone Surface, with the exception of the underside of the Anthony Henday Drive LRT Bridge which is classified XD2.

4-3.7 GEOMETRICS

A. This Section 4-3.7 [*Geometrics*] sets out minimum geometric requirements for Transportation Structures.

4-3.7.1 Structure Headslopes

A. If headslopes are used at the end of a Transportation Structure, the top of headslope widths must be the out-to-out Structure end width plus at least 2 m. The top of headslope width must be transitioned at 30:1 or flatter, back to the top of approach fill width away from the Transportation Structure. Headslopes must be no steeper than 2H:1V or the slope of the headslope of the existing bridge, whichever is flatter.

B. The minimum horizontal distance from the toe of a Transportation Structure headslope to the face of curb or edge of pavement of the under-passing Roadway must be a minimum of 3.0 m unless otherwise specified in Section 4-1 [*Description of Infrastructure*] of this Schedule.

4-3.7.2 Horizontal Clearances

A. Transportation Structures carrying Light Rail Vehicles must have adequate deck widths to accommodate the Vehicle Running Clearance Envelopes of the vehicles on two side by side tracks, emergency egress requirements in accordance with NFPA 130 and all functional requirements of the High Floor Operations and Maintenance Parameters.

B. The minimum distance from the face of a substructure element, e.g., abutment, pier, retaining wall, etc., to the adjacent roadway must be equal to, or greater than:

   1. the clear zone specified in TAC; and

   2. the offset distance required to meet the upper limit sight distance requirements as specified in TAC.

C. The use of barriers to reduce clear zone dimensions is not permitted unless otherwise specified in Section 4-1 [*Description of Infrastructure*] of this Schedule.

4-3.7.3 Vertical Clearances

A. Transportation Structures carrying Light Rail Vehicles and work trains must have adequate vertical clearances over the Trackway as defined in Chapter 3 of the HFDG.

B. The minimum vertical clearance between a sidewalk or SUP and the underside of any Transportation Structure must be 3.5 m unless otherwise specified in Section 4-1 [*Description of Infrastructure*] of this Schedule.

C. Vertical clearance signs must be provided on all Transportation Structures at the locations of under-passing Roadways or SUP and will be mounted on the lower half of the upstream fascia girder. Shop drilled holes for steel girders or cast-in inserts for concrete girders must be incorporated during girder fabrication.

4-3.8 GEOTECHNICAL

4-3.8.1 General

A. This Section 4-3.8 [*Geotechnical*] sets out general geotechnical requirements for Building Structures, Transportation Structures, Trackway, and Other Structures.

B. The geotechnical designs required for Transportation Structures, Building Structures, Trackway, and Other Structures, including foundations, slope stability assessments and stabilization, must be undertaken taking into account the geotechnical conditions encountered at the relevant Site. Site specific geotechnical investigations must be carried out in sufficient detail to allow for the identification, consideration and incorporation in the design of all geotechnical issues that may affect the performance of the Transportation Structures, Building Structures, Trackway and Other Structures throughout their Design Life.

C. The Design and Construction of all Structures must ensure the stability of all slopes, including the Blackmud Creek headslopes and sideslopes beneath the approaches, the headslopes and sideslopes of the approach fills of new Transportation Structures, and the headslopes and sideslopes of the approach embankments throughout the Design Life of the structures. All measures required to prevent erosion of embankment slopes and any altered natural slopes must be implemented.

D. The geotechnical investigations and the geotechnical engineering evaluations for Transportation Structures must be completed in accordance with the requirements of CAN/CSA S6 and the CFEM. For Building Structures and Other Structures, the geotechnical investigations and evaluations must be completed in accordance with the requirements of the NBCAE and the CFEM.

E. The use of geotechnical resistance factors corresponding to “High Degree of Understanding” per Table 6.2 of CAN/CSA S6 in the design of Transportation Structures must only be used when accepted by the City in its discretion.

F. Detailed geotechnical reports documenting the geotechnical conditions and engineering recommendations to address Design and Construction Requirements must be submitted to the City as part of the final Design Data of each Transportation Structure and Building Structure. The geotechnical assessments for Other Structures may be compiled in one or a series of reports, as may be deemed practicable by the Designer.

G. The Design-Builder must carry out additional geotechnical boreholes and testing to supplement the borehole information provided in the Disclosed Data such that the combined existing and additional information include at least one borehole at all Transportation Structure substructure locations that extend a minimum of 3 m below the estimated pile tip elevation.

H. For lineal structures such as retaining walls and Trackway, the Design-Builder must carry out additional geotechnical boreholes and testing to supplement the borehole information provided in the Disclosed Data such that the combined existing and additional information include at least one borehole every 200 m of the length of the structure.

I. For Building Structures, the Design-Builder must carry out additional geotechnical boreholes and testing to supplement the borehole information provided in the Disclosed Data such that the number and depth of the combined existing and additional boreholes at each Building Structure location are consistent with the data requirements for the foundation designs.

J. The selection of representative or “characteristic” geotechnical parameters used to determine foundation capacity must be based on the results of field and laboratory investigations appropriate to the nature of the Structure and ground conditions and must represent a cautious “best estimate” of the mean values of each parameter, taking into account all the factors that may have influence on the soil properties, in accordance with the CFEM, Section 8.5, and CAN/CSA S6.

K. The competency and frost heave susceptibility of subgrade soils beneath grade-supported Infrastructure must be evaluated as part of the final Design Data of the Infrastructure. The assessment of frost heave susceptibility must be conducted in accordance with the criteria outlined in the CFEM. Mitigation measures (e.g., soil replacement, soil reinforcement, sub-drainage, insulation, grading, surface drainage, preloading, surcharging, etc.) must be implemented as required to improve subgrade conditions, prevent frost heave, and ensure no measurable deterioration in the performance of the Infrastructure over their Design Life. The results of the geotechnical assessment and the engineering recommendations to address Design and Construction Requirements must be submitted to the City as part of the final Design Data of the Infrastructure.

4-3.8.2 Foundations

A. Foundations of Transportation Structures must be designed in accordance with CAN/CSA S6 and the CFEM. Foundations for Building Structures and Other Structures must be designed in accordance with the NBCAE and the CFEM.

4-3.8.3 Fills

A. Silt material specified as “ML” or “MH” material (in accordance with the “Modified Unified Soil Classification System” as described in the document “Prairie Farm Rehabilitation Administration, 1992, Small Dam Design and Construction Manual, Agriculture Canada, Prairie Resources Service”) must not be used in the construction of any Transportation Structure headslopes or approach fills, or in the construction of any embankments supporting the Trackway.

4-3.8.4 Slopes and Retaining Walls

A. Unless noted otherwise in Section 4-3 [*Design Guidance/Requirements*] of this Schedule the global stability of Transportation Structure headslopes, embankment sideslopes, cut slopes, and retaining walls, must be designed for a minimum factor of safety of 1.3 at the end of slope or wall construction and 1.5 in the long-term upon dissipation of construction induced excess pore water pressures. Irrespective of the computed value of the factor of safety, the inclination of permanent, unreinforced cut or fill slopes must not be steeper than 3H:1V, unless noted otherwise in Section 4-3.7 [*Geometrics*] or Section 4-3 [*Design Guidance/Requirements*] of this Schedule. The minimum factor of safety for the natural sideslopes at the Blackmud Creek Bridge location is also 1.5.

B. The design of retaining walls and Transportation Structure headslopes and approach fills must account for global stability, bearing capacity (where applicable), long-term settlements, and lateral wall deformations. Stability analyses to confirm that all headslopes and retaining walls have short-term and long-term stability sufficient to prevent failure or excessive deformation must be carried out. Deformations of the headslopes or retaining walls (including settlements and lateral movements) must be determined using appropriate deformation analyses, with representative soil parameters derived from site specific geotechnical investigations and local experience. The estimated range of embankment and wall displacements including settlements and lateral movements must be considered in the design of the Transportation Structure and must provide for acceptable structural performance and aesthetics of the approach fills and walls.

C. The geotechnical investigations and design of reinforced soil slopes must be undertaken in accordance with the requirements of FHWA-NHI-10-025 and the CFEM.

4-3.9 DRAINAGE

A. This Section 4-3.9 [*Drainage*] sets out drainage requirements for Transportation Structures unless otherwise specified. Drainage of Transportation Structures must be in accordance with this Section and Schedule 5, Part 3, Section 3-3 [*Stormwater Management*].

B. Drainage on Transportation Structures must channel all water off and away from the Transportation Structures and into the overall Stormwater Management System in a controlled manner without creating erosion, flooding, icing of Roadways, sidewalks or SUPs or other detrimental effects.

C. Drainage for Building Structures to be in accordance with Part 5 of this Schedule.

4-3.9.1 Wash Slopes/Drip Grooves

A. The tops of sidewalks, SUPs and medians adjacent to a Transportation Structure deck must have a minimum cross slope of 2%.

B. The tops of all horizontal surfaces, (i.e., abutment seat, pier cap, curb, coping cap, retaining wall, and barrier top) must have a minimum wash slope of 3%.

C. Continuous drip grooves must be provided near the outside edges of all deck soffits and top flanges of concrete box girders or trough girders so as to prevent water running down the exposed girder faces.

4-3.9.2 Drainage Collection on Decks

A. Drainage must be collected at the low points of Transportation Structure decks, and at other locations along the length of the decks as necessary, to channel water off the decks. Differential settlements and other movements between Transportation Structures and approach fills must not compromise the collection of drainage. Discharge point(s) must extend below the soffit of girders sufficient to avoid spray onto the girder.

B. Discharge point(s) of Transportation Structure drainage must be kept a minimum of 8 m away from the foundations of piers, abutments or other structure elements, and must not be directed onto any Roadways, sidewalks or SUPs.

4-3.9.3 Drainage Collection in Box Girders

A. Box girders must have 50 mm minimum diameter ventilation/drain holes provided on each side of the bottom flange at a maximum spacing of 15 m to drain water from the box girder. Additional drains must be provided wherever water can be trapped within the girder, including against internal barriers such as diaphragms, post-tensioning anchorage blisters or ribs.

4-3.9.4 Drainage Collection at Abutments/Retaining Walls

A. Joints around abutments, abutment approach slabs and retaining walls must be sealed at the surface to prevent water infiltration. A secondary system must be provided to collect, channel and remove any seepage that penetrates the seals.

B. Drainage must be provided behind abutments and retaining walls to prevent the buildup of water pressures.

C. If the material behind the abutment/retaining wall is backfill, the drainage system must include clean granular material with a maximum aggregate size of 25 mm and a maximum fines content (soil particles finer than 0.08 mm) of 5%, complete with perforated weeping drains daylighted or connected to a discharge point for Positive Drainage.

D. If the material retained behind the abutment/retaining wall is in-situ soil, the drainage system must include a sheet drain that is placed directly against the excavation face and is continuous from the top to the bottom of the wall. At the bottom of the wall the sheet drain must be connected to a perforated weeping drain located below grade and daylighted or connected to a discharge point for Positive Drainage.

E. Any buried structural element surface, other than a Splash Zone Surface, that may be exposed to leakage of salt contaminated moisture must be protected by an impervious waterproofing membrane.

F. Swales must be provided behind the tops of all retaining walls to collect and discharge surface water away from the walls in a manner that prevents erosion. The bottom and sides of swales must be lined with an impervious material to prevent infiltration of surface water into soil retained behind the walls. Swales and top of walls must slope away from abutments. The swale must be designed such that the greatest depth of drainage flow must be located at the centre of the swale. The backside of the retaining wall must not be used as part of the swale.

4-3.9.5 Drainage Collection at MSE Retaining Walls

A. Surface drainage must be controlled and channeled away from the back of MSE wall panels and the mechanically stabilized earth mass.

B. Weeping drains must be provided near the front and back bottom corners of all mechanically stabilized earth masses. The weeping drains must be daylighted or connected to a discharge point for Positive Drainage. The water level within the mechanically stabilized earth mass must be assumed at the invert level of the weep drains or higher should the design warrant it in accordance to Good Industry Practice.

C. All steel soil reinforcement located within 50 m of a roadway must be protected from exposure to de-icing salts by an impermeable membrane which must be:

   1. placed below the surface exposed to de-icing salts and above the top layer of soil reinforcement to collect and discharge all run-off;

   2. sealed to prevent leakage;

   3. sloped to drain away from the MSE wall into an intercepting weeping drain leading away from and daylighted beyond the MSE mass. The weeping tile must not be located over the steel soil reinforcement or within the reinforced soil zone;

   4. provided with a non-woven geotextile filter fabric layer that must be placed below and above the membrane to prevent puncture; and

   5. extended for the full width of the surface exposed to de-icing salts plus a minimum of 2 m beyond it on either side, and a minimum of 500 mm beyond the ends of the reinforced soil zone.

D. All joints must be shingled in the direction of drainage and welded or bonded to prevent leakage.

E. For MSE wall abutments, the concrete walkway provided in front of the abutment for inspection purposes must be underlain by the impermeable membrane.

4-3.9.6 Drainage of Soil Nail Walls

A. Surface drainage must be controlled and channeled away from the soil nail wall assembly.

B. A drainage system consisting of geocomposite drain strips, PVC connection pipes, soil nail wall footing drains, and weepholes must be provided to collect and direct perched groundwater and/or infiltrated surface water away from the soil nail wall, and to prevent the buildup of water pressure behind the wall facing. The geocomposite strip drain must have sufficient capacity to convey the anticipated water flow, and sufficient resistance to prevent collapse during construction and throughout the Design Life.

4-3.10 DUCT BANKS

A. Duct bank accommodation requirements for Transportation Structures must be allow sufficient space for all the electrical, mechanical, signals, communications and other requirements of the other sections of this Schedule.

4-3.11 INSPECTION ACCESS

A. This Section 4-3.11 [*Inspection Access*] sets out general inspection access requirements for Transportation Structures in addition to the requirements of CAN/CSA S6.

B. Transportation Structure components that are not completely accessible using conventional and readily available inspection equipment, such as manlifts and bridge inspection vehicles, must be provided with permanent access suitable for safe hands-on inspection activities.

C. For abutments on headslopes a minimum 0.6 m wide bench must be provided in front of the abutment seat suitable for bearing inspection access. The abutment height must be such that the bearings can be viewed by an inspector standing directly on the bench. The top of the abutment seat must be a maximum of 1.2 m above the bench.

D. For abutments behind retaining walls a minimum 1.0 m wide concrete inspection walkway with a 1.2 m minimum vertical clearance suitable for inspection access must be provided in front of the abutment seat. The inspection walkway must be accessible from the side without the need of any equipment but will be secured to prevent access by unauthorized persons. The top of the abutment seat must be a maximum of 1.5 m above the inspection walkway.

E. A suitably flat area must be provided at the base of all retaining walls over 2 m in exposed height to enable ladder access to the wall or abutment at any location along the wall or abutment to be done in a safe manner.

F. Voids inside abutments or other substructure must be accessible for inspections via access hatches in the abutment backwall, the hatches will be secured to prevent entry by unauthorized persons.

G. All girders having internal voids larger than a diameter of 1200 mm must:

1. have internal voids that are continuous along the length of the girder with a minimum of two access openings, openings must be secured to prevent access by unauthorized persons;

2. be provided with access openings with minimum dimensions of 820 mm by 1100 mm if rectangular and a minimum diameter of 920 mm if circular:

   a. near each girder end; and

   b. provided with access hatches that must be lockable and operable by one person.

H. Hatches for access into voids within abutments and superstructures must be situated such that a fall and/or trip hazard is not present upon entry. A platform and/or landing for access and egress may be provided within the void for this purpose. Hatches must be lockable from inside and outside the structure. Common keying must be provided for hatches on all Transportation Structures.

4-3.12 FOUNDATIONS

4-3.12.1 General

A. Foundation requirements must apply to Building Structures, Transportation Structures, and Other Structures unless noted otherwise.

B. Driven piles are not permitted anywhere except for locations within:

1. the Alberta Transportation TUC:

a. at least 100 m from the nearest residential or commercial property; and

b. greater than 20 m from any ATCO pipeline or facility

2. the approaches of the Blackmud Creek LRT Bridge.

C. The pile load carrying capacities must be determined based on the geotechnical parameters and method of installation.

D. The pile driving criteria for driven piles must be determined using wave equation analyses and verified using PDA testing.

E. All welded pile splices whose tensile or flexural capacity is required for the structural stability of a Structure must be identified on the applicable final Design Data and tested for Deficiencies using non-destructive ultrasonic testing techniques as specified in Section 4-4.3.5.5 [*Steel Pile Splices*] of this Schedule.

F. Deficiencies which are discovered must be repaired and the suspect area re-inspected.

G. Dynamically compacted, cast in-place concrete piles are not permitted.

H. Timber piles are not permitted.

I. The top of foundations for Transportation Structures, including footings and pile caps, must be buried a minimum of 600 mm below finished grade with the exception of the Anthony Henday Drive LRT Bridge.

J. Combining frost protected shallow foundations and deep foundations to support a single Transportation Structure or Building Structure is not permitted.

K. The following Structures must be supported on pile foundations:

1. Twin Brooks Station

2. Heritage Valley North Station

3. Utility Complexes and TPSS Building

4. Llew Lawrence OMF

L. An instrumentation program must be implemented to monitor the magnitude and rate of settlement of the Trackway and any structural fills placed to raise the site grade for Llew Lawrence OMF.

M. The instrumentation program must be extensive enough to provide a complete picture of fill and Trackway settlement over time.

N. Settlement monitoring must be carried out for all fills greater than 1 m thick. The quantity and spatial distribution of settlement plates is the responsibility of the Design-Builder's geotechnical engineer.

O. The monitoring instruments must be installed at critical locations/depths that allow the measuring of maximum values of settlement.

P. The monitoring results must be used to confirm compliance with the settlement criteria.

Q. Instrumentation monitoring must be carried out as follows:

1. During Construction of Llew Lawrence OMF, once every 2 weeks or more frequently if deemed necessary by the Design-Builder's geotechnical engineer based on the monitoring results and Good Industry Practice.

2. After Construction Completion and until Final Completion, every 2 months or more frequently if deemed necessary by the Design-Builder's geotechnical engineer based on monitoring results and Good Industry Practice.

R. Submit monitoring results to the City within one week of the date of measurements.

S. Helical piles are not permitted to support any portion of a Transportation Structure, or Building Structure including Stations, Utility Complexes, TPSS Building and Llew Lawrence OMF Building.

4-3.13 ABUTMENTS

4-3.13.1 General

A. Abutments must be supported on piles.

B. Wingwalls generally parallel to the approach fill must extend a minimum distance of 0.6 m beyond the top of approach fill headslope.

C. Any abutments with wingwalls over 8 m in length must include roof slabs supported on piles at both ends spanning between the end of the main superstructure members and the top of the approach fill headslope.

D. Expanded polystyrene foam and MSE walls must not be used behind abutment seats to reduce earth pressures on the abutments or abutment wingwalls or to support the abutment roof slab.

E. Abutment seats and wingwalls must be embedded a minimum of 0.5 m below finished grade.

F. Headslopes for Transportation Structures spanning over Roadways must be covered with concrete slope protection unless otherwise specified within this Schedule. The concrete slope protection must extend from the abutment seat to the bottom of the headslope and 0.5 m past the edges of the Transportation Structure on each side of the Transportation Structure.

G. Approach slabs must be used at the end of abutments. The approach slabs must be a minimum of 6 m long and settlements must not cause rotations at the abutment the approach slab is supported on that cause deviations to the track profile greater than 15 mm measured over 9.4 m.

H. Abutment designs must account for the potential horizontal movement of embankments or retaining walls.

4-3.13.2 Integral Abutments

A. Integral abutments, including both fully integral and semi-integral abutments, must not be used for bridges with the following conditions:

1. thermal spans greater than 45 m for steel girder bridges, or greater than 60 m for concrete girder bridges, where the thermal span is the distance between the thermal fixity point and the centreline of the integral abutment piles;

2. bridge abutments with a skew angle greater than 23°.

B. In addition to the requirements of Section 4-3.13.1 [*General*] of this Schedule, integral abutments must be designed to meet the following requirements:

1. The effects of skew and potential for twisting of the superstructure in plan and bi-axial bending of the piles must be analyzed and accounted for.

2. The amount of Transportation Structure and earth that have to move with the abutment during thermal movement of the superstructure must be minimized by limiting the abutment seat height above grade to not more than 1.5 m. Turned back wingwalls are required and must be parallel to the approach fill and cantilevered off the back of the abutment. Such wingwalls must not exceed 8 m in length measured from the back of the abutment seat/abutment diaphragm to the end of the wingwall.

3. Deck reinforcement must be provided at the abutments to resist negative bending moments due to torsional restraints provided by the stiff abutment diaphragms and adjacent girders.

4. For fully integral abutments (monolithic connection between abutments and superstructure) the abutment foundation must be a single row of steel H-piles. For thermal spans exceeding 22.5 m for steel girder bridges and 30 m for concrete girder bridges or when surrounding soils will restrict pile movement the piles must be installed in permanent steel casings. The casings must be filled with expanded polystyrene beads having a nominal diameter of 5 mm or an equivalent material and must be designed to allow free movement of the piles within the casings and to last for the Design Life of the supported Transportation Structure. A sacrificial corrosion thickness or galvanizing must be provided for the casings to achieve the Design Life. The H-piles must be embedded a minimum of two pile widths into the abutment seat.

   a. For fully integral abutments without driven steel piles, W-shape or built up steel sections may be used.

5. Cycle control joints at the ends of the approach slabs must be located at least 1.125 m beyond the ends of the wing walls by extending the length of the approach slab. A sleeper slab must be provided under the approach fill end of the approach slab. Drainage must be provided beneath the joint between the approach slab and sleeper slab to drain water away from the abutment.

6. Approach slabs must not move longitudinally in and out between stationary and parallel non-integral wingwalls.

7. Two layers of polyethylene sheet must be provided under the approach slab to minimize frictional forces due to longitudinal movement. The connection between the approach slab and the superstructure must be designed to resist all friction forces due to horizontal movement.

8. Barriers constructed on approach slabs must be designed such that:

   a. loss of barrier height due to settlement does not exceed 30 mm;

   b. the differential settlement between adjacent barrier segments does not exceed 25 mm; and

   c. the joints between barrier segments remain sealed.

9. Provision must be made to accommodate thermal movement between integral abutments, slope protection, inspection walkways, etc. gaps must be protected against moisture ingress.

4-3.14 PIERS

A. The ends of pier cap cantilevers must have cast-in stainless steel drip sheets across the full underside width of the pier cap or equivalent to prevent staining of substructure concrete.

4-3.15 RETAINING WALLS

4-3.15.1 General

A. Retaining walls parallel to a Roadway located adjacent to the base of the wall must have their ends on the approaching traffic side flared away from traffic at a 20:1 taper. A 20:1 taper must also be used for both ends of walls parallel to a Trackway located adjacent to the base of the wall.

B. Lateral displacements of the tops of retaining walls, over the Design Life of the structure, must not adversely affect the safety, serviceability and durability of:

1. the Infrastructure, including the retaining wall; and

2. any buildings, surface facilities and Utility Infrastructure.

C. Notwithstanding Section 4-3.15.1.B [*General*] of this Schedule, lateral displacements, over the Design Life of the structure, of the tops of retaining walls must in no case exceed 20 mm.

D. The exterior faces of retaining walls must be sloped at a minimum of 1H:50V towards the retained soil and must be designed to discourage attempts to climb the wall.

E. The top of retaining walls, including copings, must have a consistent negative slope from its high point to the ends of the wall.

F. Any Transportation Structure components located immediately behind retaining walls, including abutment seats, abutment wingwalls, abutment deck joints, abutment bearings and barriers, must be designed to accommodate any movements resulting from retaining wall displacements.

G. Dry cast concrete block walls or stacked masonry block walls are not permitted for retaining walls exceeding 1.2 m in height, except for the retaining wall on the west side of 111 Street NW and north of 23 Avenue NW.

4-3.15.2 Mechanically Stabilized Earth Walls

A. All MSE walls must comply with the requirements of CAN/CSA S6 except that the capacity of the MSE wall must be determined in accordance with AASHTO LRFD.

B. MSE walls must not be used for Transportation Structures crossing watercourses.

C. Maximum reinforcement loads must be calculated using the “Simplified Method” as presented in AASHTO LRFD.

D. MSE wall embedment depths below finished grade must not be less than the minimum depth provided in Table C11.10.2.2-1 “Guide for Minimum Front Face Embedment Depth” in the AASHTO LRFD Commentary, but must not be less than 1 m.

E. MSE wall backfill must extend a minimum of 0.5 m beyond the end of the soil reinforcement.

F. Mechanically stabilized earth walls are not permitted in locations where the wall is required to support a Structure.

4-3.15.3 Utilities

A. Mechanically stabilized earth must not be placed over or in the vicinity of any Utility Infrastructure, unless the following conditions are met:

1. all applicable Utility Infrastructure can be removed and repaired without disturbing the mechanically stabilized earth;

2. Utility Infrastructure carrying potentially eroding materials, including water carrying appurtenances, such as catch basins, drainage inlets/outlets, and culverts, are not permitted within 10 m of any MSE wall backfill unless the Utility Infrastructure are appropriately cased to protect the MSE wall system from any leakage, and the extent of the casing is sufficient to protect the MSE wall system against discharges from the ends of the casing; and

3. no change of direction of Utility lines, and no valves, valve chambers or other discontinuity is permitted within the mechanically stabilized earth.

4-3.15.3.1 Facing

A. All MSE walls must be faced with precast concrete wall panels. The minimum precast concrete panel thickness must be 140 mm, excluding any additional thickness required for aesthetic surface treatment.

B. The precast concrete panel system must not be subjected to a differential settlement of more than 100 mm in 10 m of length along the wall. For MSE walls with full height precast concrete panels, the total settlement must be limited to a maximum of 50 mm, and the differential settlement must not exceed 20 mm in 10 m of length along the wall.

C. Joints between panels must prevent the loss of fill through the joints.

D. Corner units must be provided and designed to prevent joint gaps from opening up between adjacent panels orientated in different directions. Acute wall corners less than 70° (measured between backfill sides of panels) must not be used.

E. The non-exposed side of MSE wall panels must be in full contact with compacted backfill.

F. Installed MSE wall panels must be repairable/replaceable without adverse impact to the Transportation Structure. A repair/replacement procedure must be submitted to the City with the applicable final Design Data.

4-3.15.3.2 Coping Cap

A. A cast-in-place concrete coping cap must be placed on the top of all MSE walls not covered by a concrete barrier and must have full depth joints lining up with panel joints.

B. The top of the cast-in-place concrete wall coping must be smooth and have no steps or abrupt changes in height. The top of the coping must have a consistent negative slope from its high point to the ends of the wall.

C. Copings must have control joints perpendicular to the wall alignment. The spacing of the control joints must not exceed 4 m. Longitudinal steel in the copings must be discontinuous and have 50 mm cover measured from the centre of the control joint. The copings must also have drip grooves in the soffit.

D. Typical control joint details must be in accordance with the Control Joint Detail in the Alberta Transportation Standard Drawing S1412-20.

4-3.15.3.3 Barriers

A. MSE walls with traffic running adjacent to the top of the wall must have rigid traffic barriers. Such barriers must be supported on moment slabs to resist sliding and overturning and must be located on top of the MSE walls. Flexible guardrail systems must not be used. The MSE wall must be designed to resist the loads applied to the barrier.

B. MSE walls with a sidewalk or SUP adjacent to the top of the wall must be provided with a pedestrian or bicycle rail as required by CAN/CSA S6. The rail must be mounted on the top surface of the concrete coping of the MSE wall.

4-3.15.3.4 Obstructions within the Backfill

A. Soil reinforcing must accommodate any obstruction within the mechanically stabilized earth, including foundation piles and associated casings, and casings for future pile installations. For MSE wall systems that lend themselves to splaying of the soil reinforcement, the splay angle must not exceed 15° perpendicular to the facing panel. For MSE wall systems that do not lend themselves to splaying, additional soil reinforcement must be provided to compensate for the loss of soil reinforcement at obstruction locations.

4-3.15.3.5 Inspection Wires

A. Galvanized steel inspection wires must be provided in all steel reinforced MSE wall systems in addition to the soil reinforcement design requirements. One inspection wire must be provided for each 25 m^2 of wall area. Inspection wires must be placed in vertically distributed sets of two or three depending on the wall height. Two locations must be provided where the wall height is less than 6 m and three locations provided where the wall height is greater than 6 m. Vertical distribution must be such that a single inspection wire is placed within the centre of the bottom wall panel, centre of the top wall panel, and in the centre wall panel where three locations are required. Sets of inspection wires must be evenly distributed along the length of the wall.

B. Inspection access ports and wire removal and centering devices must be detailed in accordance with the California Department of Transportation standard bridge detail sheet XS13-020-3. Inspection access ports must be cast as voids in the panels at the panel manufacturing facility and the remaining cavity placed and filled with an OH-V patching product from the Alberta Transportation Products List and in accordance with the manufacturer’s recommendations. All inspection access ports must be marked with a 25 mm diameter galvanized survey target anchored into the patching material and flush with the wall surface. Adhesively mounted survey targets will not be permitted.

4-3.15.4 Retaining Walls with Ground Anchors

A. Ground anchors must not extend laterally beyond the boundary of the City Lands.

B. Ground anchors must be designed in accordance with the most stringent requirements of the following standards:

1. PTI DC35.1
2. CAN/CSA S6

C. The load carrying capacity of ground anchors must be verified by verification tests on sacrificial pre-production anchors and performance and proof tests on production anchors, in accordance with the recommendations of PTI DC35.1. For permanent anchors, a minimum of one verification test in each significantly different soil condition (in terms of geologic origin, composition and strength) at each Transportation Structure must be performed. Performance tests must be conducted on a minimum of 5 percent of production anchors, and proof tests must be carried out on all production anchors not subjected to performance tests.

D. The test setup, testing procedures and results of verification tests must be reviewed and accepted by the Designer and provided to the City prior to the installation of production ground anchors.

E. The minimum bond length of a ground anchor must be 4.5 m.

F. The factored design load, service design load, lock-off load, and test load of the ground anchors must be stated on the applicable final Design Data.

G. The free stressing length (unbonded length) of a ground anchor must extend at least 1.5 m or 20% of the height of the wall, whichever is greater, behind the critical failure surface. The critical failure surface must be determined using slope stability analyses.

H. A ground anchor design report containing all design parameters required for load resistance calculations, installation procedures, procedures required for installation verification, results of verification and proof tests, and details on how the corrosion protection system provides the necessary corrosion protection over the Design Life must be submitted to the City with the applicable final Design Data.

I. Ground anchors must be in accordance with Section 4-4.5 [*Ground Anchors*] of this Schedule.

4-3.15.5 Soil Nail Walls

A. Soil nails must not extend laterally beyond the boundary of the City Lands.

B. Soil nail walls must be designed in accordance with the requirements in FHWA-NHI-14-007.

C. The pullout capacity of soil nails must be verified by verification tests on sacrificial pre-production soil nails and proof tests on production soil nails, in accordance with the recommendations of FHWA-NHI-14-007. For permanent soil nails, a minimum of one verification test in each different soil condition (in terms of geologic origin, composition and strength) must be performed at the location of each soil nailed structure. Proof tests must be conducted on a minimum of 5% of production soil nails in each nail row with a minimum of one test per row. Verification tests must not be counted towards the number of required proof tests.

D. The test setup, testing procedures and results of verification tests must be reviewed and accepted by the Designer and provided to the City prior to the installation of production soil nails.

E. The factored design load, service design load, and test load of the soil nails must be stated on the applicable final Design Data.

F. A soil nail wall design report containing all design parameters required for load resistance calculations, installation procedures, results of verification and proof tests, and procedures required for installation verification must be submitted to the City with the applicable final Design Data.

G. Soil nail tendons must be in accordance with Section 4-4.17 [*Soil Nails*] of this Schedule.

4-3.16 BEARINGS

4-3.16.1 General

A. Expansion bearings must provide an excess travel capacity in each direction of at least 25% of the theoretical thermal movement, but not less than 25 mm. An allowance must be made for additional movement including movements due to concrete creep, shrinkage, and foundation or embankment movements.

B. Steel sole plates and base plates must be provided. All steel components except those welded to steel girders must be galvanized, metallized, or stainless steel.

C. The beneficial effect of friction must be neglected in proportioning fasteners and anchors to resist horizontal loads at the ULS.

D. An 80 mm nominal thickness grout pad must be provided under all bearing base plates. The grout must sit in a grout pocket recessed 40 mm nominally into the top of the substructure. The grout pocket must be at least 75 mm larger than the base plate around the perimeter.

E. The concrete surrounding the grout pocket recess must be cast monolithically with the supporting substructure element.

F. Shim plates used for shim stacks must be hot-dip galvanized.

G. Attachment of bearing sole plates to steel girders by welding must be in the longitudinal direction along the edge of the girder.

   1. If an elastomer is used, the weld must be a minimum of 40 mm away from the elastomer.

   2. Overhead welding is not permitted.

   3. Transverse sole plate ends not welded must be sealed against moisture.

H. Sole plates attached to concrete girders and base plates attached to concrete substructure must be sealed against moisture.

I. Bearings must be designed and detailed to allow for bearing replacement without damage to the Transportation Structure and without removal of any concrete, welds, or anchorages permanently attached to the Transportation Structure. Bearing replacement must be designed based on simultaneously jacking all girder lines and supporting them in the raised position while bearings are replaced one at a time. Bearings must be replaceable with a maximum jacking height of 5 mm. Future jacking locations must be located on the permanent substructure and must not require the installation of temporary supports, use of proprietary equipment, or require prolonged traffic closures. Locations for future jacking must be shown on the applicable final Design Data and must be based on estimated jack and distribution plate sizes. Details of the bearing replacement procedure must be included in the Interim Design Submittal and noted on the applicable final Design Data, together with the unfactored dead load and live load jacking forces that will be required for bearing replacement.

J. Disk bearings must not be used.

K. The height of each bearing base plate must be adjustable until after the superstructure is erected and installed on to the bearing.

L. Uplift of bearings is not permitted at serviceability limit states.

4-3.16.2 Elastomeric Bearings

A. Elastomeric bearings must incorporate the following standard features:

   1. Elastomeric bearing pads must be designed at SLS for all rotations that take place after the bearings are grouted, plus a tolerance of 0.005 radians. Rotations taking place prior to grouting need not be considered if the bearing base plate is supported on a self-rocking pintle that ensures uniform contact between the elastomeric bearing pad and the bearing sole plate/girder bottom flange at erection.

   2. Notwithstanding Section 11.6.6.2.2 of CAN/CSA S6, material requirements for elastomers must conform to AASHTO M251 Standard Specification for Plain and Laminated Elastomeric Bridge Bearings (AASHTO M251). Cured elastomeric compounds must be low temperature Grade 5 and meet the minimum requirements listed in Table X1 of AASHTO M251 with a Shore A durometer hardness of 60. For fully integral abutments and piers, cured elastomeric compounds must be Shore A durometer hardness of 50.

3. Elastomeric bearing pads must be designed for a maximum of 10 mm of horizontal deformation unless a sliding surface is provided which can accommodate the total design movement at the bearing.

4. Sliding surfaces must allow for translation by sliding of a stainless steel surface against a mating PTFE element. PTFE must be 4.8 mm thick unlubricated, unfilled 100% virgin polymer conforming to Section 18.8.2.5 (*Unfilled PTFE Sheet*) of the AASHTO LRFD Bridge Construction Specifications (AASHTO LRFD BCS). PTFE sheets must be recessed and bonded into a 2.5 mm deep recess in the top of a minimum 10 mm thick galvanized steel plate vulcanized to the top of the elastomeric pad. PTFE sheets must have the same plan dimensions as the elastomeric pad.

5. Elastomeric bearing pads must be restrained from walking out by means of keeper bars attached to the top of the base plate.

4-3.16.3 Pot Bearings

A. Pot bearings must incorporate the following standard features:

1. Bearings must be designed to prevent moisture and dirt from entering internal surfaces.

2. Expansion bearings must allow for translation by sliding of a stainless steel surface against a mating PTFE element. Except for lateral restraints, the stainless steel surface must be positioned above the PTFE element.

3. Except when used as a mating surface for guides for lateral restraints, PTFE must be unfilled, 100% virgin polymer conforming to Section 18.8.2.5 (Unfilled PTFE Sheet) of the AASHTO LRFD BCS and contain spherical reservoirs for lubricant pressed into its surface. The diameter of the reservoirs must not exceed 8 mm measured at the surface of the PTFE, and the depth must not be less than 2 mm nor more than half the thickness of the PTFE. The reservoirs must be evenly distributed across the surface of the PTFE and must occupy 20% to 30% of the surface.

4. PTFE used as a mating surface for guides for lateral restraint must not be dimpled or lubricated. All PTFE elements must be fully bonded and recessed in a rigid backing material.

5. All PTFE surfaces except those that act as mating surfaces for guides for lateral restraint or that are subjected to a contact pressure of less than 5 MPa must be permanently lubricated with silicone grease.

6. Notwithstanding Section 11.6.3.6 of CAN/CSA S6, the average contact pressure for unfilled PTFE elements, based on the recessed area of the PTFE, must not exceed the valves specified in Table 4-3.16.3-1 [*Average Contact Pressure for Unfilled PTFE Elements*].

Table 4-3.16.3-1 Average Contact Pressure for Unfilled PTFE Elements

Limit State	Permanent Load (MPa)	All Loads (MPa)
SLS	25	35
ULS	40	55

7. The maximum contact pressures at the extreme edges of flat and curved PTFE elements must not exceed 1.2 times the values specified in Table 4-3.16.3-1 [*Average Contact Pressure for Unfilled PTFE Elements*].

8. Notwithstanding Section 11.6.3.6 of CAN/CSA S6, the average contact pressure for all loads on PTFE elements filled with up to 15% by mass of glass fibres and used to face mating surfaces of guides for lateral restraint must not exceed 45 MPa at SLS and 55 MPa at ULS.

9. Notwithstanding Section 11.6.5.4 of CAN/CSA S6, the average stress in the elastomer at SLS loads must not exceed 30 MPa. Notwithstanding Section 11.6.5.2 and 11.6.6.2.2 of CAN/CSA S6, cured elastomeric compounds must be low temperature Grade 5 and meet the minimum requirements listed in Table X1 of AASHTO M251 and Shore A durometer hardness of 50.

10. The elastomer must be a single disc of confined elastomer. The effective thickness of the elastomeric disk to evaluate the rotational capacity of the bearing must be limited to the thickness of the disk excluding the brass rings.

11. Pot bearings must be installed on a level base plate on galvanized steel shim stacks. The bearings must be designed for all rotations that take place at the SLS and ULS conditions, plus a fabrication and construction tolerance allowance of 0.01 radians. The total rotational capacity must not be less than ± 1°.

12. The coefficient of friction between stainless steel sliding surfaces and lubricated virgin PTFE must be in accordance with Section 14.7.2.5 and Table 14.7.2.5-1 of the AASHTO LRFD BCS.

13. The depth of the pot wall must be such that a vertical distance of at least 2.5 mm remains between the top of the pot wall and the closest point of contact of the brass sealing rings with the pot wall upon rotating the piston an amount equal to the maximum design rotation at the ULS.

14. The pot and piston surfaces in contact with the confined elastomer must be lubricated with silicone grease. The bearing must be sealed by a one piece continuous pre-formed closed cell compressible ring against entry of dirt, dust and moisture between the elastomer and the pot and piston contact surfaces. Any joint in the ring must be bonded and the strength must be at least equal to the strength of the ring.

15. Bearings must be set level by using tapered sole plates except at cover plated joints, where the sliding plane of the abutment expansion bearings must be set parallel to the grade slope for proper functioning of the joints. In this case, the effects on the Transportation Structure of longitudinal forces generated by the inclined sliding bearings must be accounted for.

16. The surfaces in contact with the elastomer must not be metallized or galvanized and must be lubricated with silicone grease.

4-3.17 GIRDERS

4-3.17.1 General

A. Continuous span Transportation Structures must have the same number of girder lines in adjacent spans or adjacent segments, such that each individual girder line is fully continuous from end-to-end of the Transportation Structure.

4-3.17.2 Concrete Girders

A. Concrete girders must meet the following requirements:

1. Stirrup projections from the top of the girder into the deck must meet CAN/CSA S6 requirements for developing full composite action between the girder and the deck. All stirrups must be hooked around longitudinal bars. When the projection of the underside of the stirrup tops is less than 25 mm above the top of the bottom mat of deck bars, additional extension bars must be provided to tie the girder and the deck together to provide composite action.

2. The horizontal interface shear design for composite action between the girder and the deck must satisfy the requirements of CAN/CSA S6 or AASHTO LRFD, whichever is more stringent. The longitudinal distribution of shear forces must be taken to be the same as the ULS applied shear envelope.

3. The area of stirrups required for end crack control in pre-tensioning anchor zones must be calculated in accordance with CAN/CSA S6, Section 8.16.3.2. Fifty percent of this amount of stirrups must be distributed over a distance equal to 0.125 h from the end of the girder where “h” is the depth of the girder. The end stirrup must be located as close to the end of the girder as cover permits.

4. For post-tensioning ducts in concrete girders with a 28-day concrete strength greater than or equal to 65 MPa, the inside duct diameter must not exceed 50% of the web thickness and the inside duct area must be greater than 250% of the strand area.

B. Concrete girders that are adjoined in parallel (i.e., side by side) must be steel reinforced through the shear interface between the girders for the entire girder length.

4-3.17.2.1 Segmental Concrete Girders

A. General

1. Segmental concrete girders must meet the following requirements, in addition to those of Section 4-3.17.2 [*Concrete Girders*] of this Schedule. The method of construction must be shown on the applicable final Design Data.

B. Loads

1. Thermal Loads

   a. In lieu of the requirements of CAN/CSA S6, Section 3.9.4.4 (Thermal gradient effects), segmental concrete girders must be designed for the temperature gradient specified for Zone 1 in AASHTO LRFD, Section 3.12.3 (Temperature Gradient).

2. Creep and Shrinkage

   a. Creep and shrinkage strains must be based on the provisions of the CEB-FIP rather than on the requirements of CAN/CSA S6, Sections 8.4.1.5 (Shrinkage) and 8.4.1.6 (Creep).

   b. The creep and shrinkage strains predicted by CEB-FIP must be adjusted as required based on tests carried out on the actual concrete mix used for the girders, including on tests measuring concrete creep and shrinkage.

3. Closure Force Loads

   a. Closure forces for segmental concrete cantilever construction due to vertical girder misalignment must be based on a minimum girder misalignment of L/1000 (where L is the cantilever length from centre of pier to the cantilever tip) and assuming uncracked sections.

   b. Closure forces must be used as load “K” in CAN/CSA S6.

C. Analysis

1. Transverse Analysis

   a. The transverse design of segmental concrete box girder segments for flexure must consider the segment as a rigid box frame. Flanges must be analyzed as variable depth sections considering the fillets between the flanges and the webs. Wheel loads must be positioned to provide maximum moments, and elastic analysis must be used to determine the effective longitudinal distribution of wheel loads for each load location. Increase in web shear and other effects on the cross section resulting from eccentric loading or unsymmetrical structure geometry must be accounted for.

b. Transverse elastic and creep shortening due to prestressing and shrinkage must be accounted for in the transverse analysis.

c. The effects of secondary moments due to prestressing must be included in stress calculations at the SLS and during Construction. Secondary moments must also be accounted for at the ULS.

2. Longitudinal Analysis

a. Longitudinal analysis of segmental concrete girders must account for the actual Construction method and Construction Schedule as well as the time-related effects of concrete creep, shrinkage and prestress losses.

b. The effects of secondary moments due to prestressing must be included in stress calculations at the SLS and during Construction. Secondary moments must also be accounted for at the ULS.

c. All Construction loads and conditions, temporary supports or restraints, closure forces due to misalignment corrections and changes in the structural static system occurring during Construction must be accounted for.

3. Analysis of Final Structural System

a. The final structural system must be analyzed and designed for redistribution of Construction stage force effects due to internal deformations and changes in support and restraint conditions, including accumulated locked-in force effects from the Construction process.

4. Analysis of Girder Segment Joints

a. Joints in segmental girders made continuous by unbonded post-tensioning steel must be designed at the ULS for the simultaneous effect of axial force, moment and shear that may occur at a joint. These force effects, the opening of the joint, and the remaining contact surface between the components must be determined by global consideration of strain and deformation. Shear must be assumed to be transmitted through the contact area only.

5. Serviceability Limit State Stresses

a. Stresses at SLS must be in accordance with CAN/CSA S6, Section 8.8.4.6, such that:

i. the principle tensile stress at the neutral axis of the girder must not exceed $0.288\sqrt{f'c}$ for SLS Load Combination 1; and

ii. the maximum concrete compression stress at SLS under permanent loads must not exceed 0.4f’c.

b. The principal tensile stress must be determined using classical beam theory and Mohr’s Circle. The width of the web for these calculations must be measured perpendicular to the plane of the web. The vertical force component of draped longitudinal tendons may be considered to reduce the shear force due to the applied loads provided the tendons are anchored or fully developed in the top or bottom 1/3 of the webs. Local tensions produced in the webs due to the anchorage of tendons must be included in the principal tension stress check.

D. Resistance Factors

1. Resistance factors for the ULS must be in accordance with CAN/CSA S6 if the post-tensioning tendons are fully bonded. If the post-tensioning tendons are partially bonded or unbonded the resistance factors must be reduced by 0.05 from those given in CAN/CSA S6. In order for a post-tensioning tendon to be considered to be fully bonded the tendons must be fully developed at the section being considered. If a bonded tendon is not fully developed at the section under consideration, it must be considered to be partially bonded.

2. Where the post-tensioning is a combination of fully bonded tendons, partially bonded tendons and unbonded tendons, the resistance factor at any section must be based on fully bonded tendons, if the tendons providing the majority of the prestressing force at the section are fully bonded and on unbonded tendons if the tendons providing the majority of the prestressing force at the section are partially bonded or unbonded.

E. Girder Detailing

1. Minimum Top Flange Thickness

   a. Girders must have a minimum top flange thickness of 200 mm except that the minimum thickness must be increased to 230 mm in anchorage zones where transverse post-tensioning is used.

2. Minimum Web Thickness

   a. Girders must have a minimum web thickness of:

      i. 200 mm if there are no longitudinal post-tensioning tendons in the webs; and

      ii. 300 mm if there are longitudinal post-tensioning tendons in the webs.

3. Closure Segment

   a. Cast-in-place concrete closure joints wider than 225 mm must be reinforced with concrete reinforcement.

4. Post-Tensioning Tendons

   a. All Transportation Structures erected using the balanced cantilever method must have a minimum of two draped external or internal continuity post-tensioning tendons per girder web that extend to the adjacent pier or abutment diaphragms.

   b. Vertical post-tensioning tendons must not be used.

   c. The unsupported length of external post-tensioning tendons must not exceed 8.0 m.

5. External Post-Tensioning Tendon Deviators

   a. External post-tensioning tendon deviators must fully extend from the bottom flange to the top flange of the girder.

   b. External post-tensioning tendons passing through deviators must be contained in grouted steel pipes cast into the deviators.

6. Internal Post-Tensioning Ducts

   a. Internal post-tensioning ducts must be positively sealed with segmental duct couplers or o-rings at all segment joints. Duct couplers must have a maximum deflection angle of 6° at the segment joints. The duct couplers must be mounted perpendicular to the bulkheads at the segment joints.

   b. The minimum centre-to-centre post-tensioning duct spacing must be the greater of 200 mm, two times the outer duct diameter and the outer duct diameter plus 115 mm.

4-3.17.3 Steel Girders

A. Steel girders must meet the following requirements:

1. In accordance with AWS D1.5, Bridge Welding Code, the bottom of the bearing ends of bearing stiffeners must be flush and square with the web and must have a minimum of 75% of this area in contact with the flanges. The top must be a “tight fit” and must have a maximum gap of 1 mm. The bearing stiffeners must be fillet welded to both the top and bottom flanges and to the web.

2. Jacking stiffeners must be provided for future bearing replacement. Locations of jacking stiffeners must be based on the estimated jack sizes required for bearing replacement, plus sufficient clearance to the edge of the abutment seat or pier cap.

3. Diaphragm connector plates as well as intermediate stiffeners at stress reversal locations must be welded to both top and bottom flanges. Intermediate stiffeners, other than at stress reversal locations, must be welded to the compression flange only, and cut short of the tension flange with a web gap meeting the requirement of Section 10.10.6.4 of CAN/CSA S6.

4. No intersecting welds are allowed. The ends of stiffeners must be corner coped a minimum of 25 mm x 25 mm.

5. If a stiffener is wider than the flange, the end of the stiffener must be coped at 45 degrees from the flange edge.

6. All weld ends for stiffeners, gussets, and other attachments to girders must terminate at least 10 mm from the edge or end of the plates.

7. Gusset plates for attachment of horizontal bracing must be bolted and not welded to girders.

8. Staining of the substructure concrete or any other Structure components beneath the girders must be prevented. Measures taken to prevent staining must include:

   a. at pier locations, as a minimum, the exterior edge of the bottom flange of exterior steel girders must have a 19 x 19 x 8000 mm long rubber strip centred over the pier; and

   b. at abutments, as a minimum, exterior steel girders must 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 fully integral abutments, a second rubber strip must be applied all around the bottom flange of all girders immediately in front of the concrete abutment face.

9. Shear stud projections from the top of girder flanges into the deck must meet all CAN/CSA S6 requirements for stud development and anchorage and ensure full composite action between the girder and the deck. When the shear stud projection, measured from the underside of the head of the stud to the top of the bottom transverse deck reinforcement, is less than 25 mm, additional reinforcement must be provided and designed as shear friction reinforcement for a horizontal shear plane at the deck/girder haunch interface.

4-3.18 DECKS

4-3.18.1 General

A. The design of Transportation Structure barriers and decks for load effects due to barrier loading may be based on the AASHTO LRFD, Appendix A13.

B. Cast-in-place concrete decks must meet the following requirements:

   1. deck slabs must have a minimum thickness of 225 mm, unless otherwise specified, and must have two mats of concrete reinforcement;

   2. deck slabs, supporting sidewalks or SUPs, must have a minimum thickness of 175 mm; and

   3. Stay-in-place deck soffit formwork will not be permitted.

C. Deck on girder slabs using precast concrete partial depth deck panels must meet the following requirements:

   1. Deck slabs using precast concrete partial depth deck panels must consist of a cast-in-place concrete deck slab on precast concrete partial depth deck panels.

   2. The cast-in-place concrete deck slab must be designed to be fully composite with the precast concrete partial depth deck panels.

   3. The minimum composite deck slab system (precast concrete partial depth deck panels and cast-in-place concrete deck together) thickness must be the greater of the girder spacing divided by 15.0 or 225 mm (i.e., the minimum allowable combined thickness of the precast concrete partial depth deck panels and the cast-in-place deck is 225 mm). In addition, the following must be satisfied:

      a. The precast concrete partial depth deck panels must have a minimum thickness of 90 mm.

      b. The cast-in-place concrete portion of the composite deck slab system must have a minimum thickness of 115 mm.

      c. The cast-in-place concrete portion of the composite slab system must have sufficient thickness to satisfy all reinforcement cover requirements and maintain adequate spacing between reinforcement bars.

   4. The precast concrete partial depth deck panels must be fully pre-tensioned and the stresses in the precast concrete partial depth deck panels must not exceed the following:

      a. From transfer until the 28 day strength is attained:

         i. Compression: 0.6 f’ci

         ii. Tension: 0.5 fcri

      b. After the 28 day strength is attained and at serviceability limit states:

         i. Tension: fcr

      c. The average compressive stress in the precast concrete partial depth deck panels at pre-tension strand release must be ≤ 7.0 MPa.

   5. The empirical design method in accordance with the CHBDC Clause 8.18.4 is not permitted for design of the composite deck slab system using precast concrete partial depth deck panels.

6. The composite deck slab system must be designed using flexural design methods. All moments must be determined by elastic analysis. For all bridges the following minimum transverse positive moment reinforcing must be provided over supporting girder lines:

    a. In addition to the required pre-tensioning strands, transverse reinforcing bars matching the same grade as those used in the cast-in-place deck, with a minimum reinforcement ratio “ρ” of 0.003, must be provided throughout the precast concrete partial depth deck panels and must project over the girder lines and into the cast-in-place concrete portion of the composite deck slab system. The reinforcement ratio “ρ” must be calculated for “d” equal to the effective depth of the composite deck slab system. The spacing of the transverse stainless steel reinforcing bars must not exceed 300 mm;

    b. At interior girder lines, the transverse stainless steel reinforcing bars must project out of the precast concrete partial depth deck panel edges and over the girder flanges as required to provide a full lap splice (i.e., adequate to develop the bar yield capacity) with the bars projecting from opposing precast concrete partial depth deck panels supported on the same girder. At exterior girder lines, the transverse stainless steel reinforcing bars must be extended at least one full development length beyond the exterior girder centreline;

7. Precast concrete partial depth deck panels must consist of Class HPC Concrete.

8. The composite deck slab system must conform to the following:

    a. The precast concrete partial depth deck panels must have a minimum age of 45 days and a maximum age of 120 days when the cast-in-place portion of the deck is cast.

    b. The cast-in-place concrete portion must have continuous bottom longitudinal reinforcing bars (parallel to girders lines) placed directly on top of the precast concrete partial depth deck panels. This reinforcement layer must utilize a minimum size of 15M bars at a maximum spacing of 300 mm on centre.

9. Pre-tensioning strands must be 9.5 mm diameter.

10. Pre-tensioning strands must not project beyond the edges of the precast concrete partial depth deck panels.

11. Pre-tensioning strands cast into the precast concrete partial depth deck panels must be uncoated steel.

12. With a steel girder superstructure, the following additional provisions apply:

    a. The precast concrete partial depth deck panel length must be set to provide a minimum 75 mm long bearing zone (as measured perpendicular to the girder line) on the haunch concrete. A minimum 50 mm thick haunch must be provided beneath the underside of the precast concrete partial depth deck panels.

    b. The girder top flange must have a minimum width of 450 mm.

    c. Shear studs attached to the girder top flange must project above the top surface of the flange to provide at least 25 mm clearance between the underside of the shear stud head and the top of the precast concrete partial depth deck panels. If this clearance is not met, additional hat shape bars will be required to ensure a composite connection between girder and concrete.

13. With precast concrete girder superstructures, the following additional provisions apply:

a. For NU girders or any other girder shape where the top flange is less than 150 mm thick at the flange edges, the precast concrete partial depth deck panel length must be set to provide a minimum 200 mm long bearing zone (as measured perpendicular to the girder line) on the haunch concrete. For all other girders, the precast concrete partial depth deck panel length must be set to provide a minimum 75 mm bearing zone (as measured perpendicular to the girder line) on the haunch concrete. A minimum 50 mm thick haunch must be provided beneath the underside of the precast concrete partial depth deck panels.

b. Stirrups projecting from the top girder flange must project above the top surface of the flange to provide at least 25 mm clearance between the underside of the stirrup tops and the top of the precast concrete partial depth deck panels.

14. The maximum allowable haunch height with precast concrete partial depth deck panels is 150 mm, as measured at the outer edge of the girder flange. If the haunch exceeds this height, additional analysis is required to determine whether the unreinforced section of the haunch (i.e., that part of the haunch supporting the precast concrete partial depth deck panel) requires additional reinforcing.

15. Vertical bleed holes must be provided through the precast concrete partial depth deck panels and evenly distributed along the two supported panel edges. The holes must be not less than 25 mm diameter and must be located adjacent to the formed edge of the haunch to facilitate the escape of entrapped air.

16. When a bridge includes a separation barrier between a sidewalk and traffic or train, any reinforcement required to anchor the separation barrier to the deck must be cast into the precast concrete partial depth deck panels and project into the barrier.

17. It is not permitted for any hardware associated with deck formwork, including deck overhang formwork, to be visible after removal of all formwork.

4-3.19 DECK JOINTS

4-3.19.1 General

A. The deck joint expansion gaps must close before the barrier expansion gaps at deck joints.

B. All deck joints must be sealed deck joints.

C. Deck joints must run continuously across the full width of the deck and must be turned up at their ends as required to prevent water from draining out of their ends.

D. Exterior barriers and curbs must have removable cover plates on the inside face and across the top.

E. Interior barriers and medians must have removable cover plates on both sides and across the top.

F. Deck joints across the width of sidewalks, SUPs and emergency egress routes must have non-slip surface cover plates.

G. The free ends of any deck joint cover plates at abutments must be fixed to the deck side to allow for jacking of the superstructure.

H. Deck joint cover plates at piers must be removable to allow for jacking of the superstructure.

4-3.19.1.1 Sealed Deck Joints

A. Sealed deck joints must include Strip Seal Deck Joints and Cover Plated V-Seal Deck Joints in accordance with the Alberta Transportation Standard Drawings listed in Table 4-3.19.1.1-1 [*Alberta Transportation Standard Drawings for Sealed Deck Joints*].

B. Deck joints must incorporate stop movement bars to maintain a minimum joint gap sufficient for seal replacement. The joint gap must be maintained at a minimum of 60 mm notwithstanding any narrower gap width recommended by the manufacturer. The maximum allowable gap for sealed deck joints must be taken as the summation of the minimum 60 mm and the maximum permissible normal movement.

C. The maximum permissible shear movement must be based on the maximum absolute temperature difference between the temperature at the time of joint installation and the maximum or minimum design temperature, whichever is greater.

D. For skew bridges, the longitudinal direction of bridge movement must be resolved into its normal and shear 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 listed in Table 4-3.19.1.1-1 [*Alberta Transportation Standard Drawings for Sealed Deck Joints*]. The normal component gap must be set at the time of concreting the joint extrusion, in accordance with the temperature setting chart provided on the standard drawings, but the shear component must be zero at the time of seal installation. The temperature of seal installation is assumed to be 15°C for the standard design but must be adjusted for the installation temperature.

E. The Type I Strip Seal Deck Joint is the preferred deck joint and must be used, unless the Design-Builder can demonstrate to the satisfaction of the City that the Type I Strip Seal Deck Joint cannot be installed.

F. Setting of deck joints may be 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 structure and 24 hours for steel structures.

G. Only neoprene seals are permitted.

H. The construction joint for deck joint blockouts in the widened roadway bridge must be located outside the width of the concrete paving lip and must be within the extents of the deck waterproofing system.

Table 4-3.19.1.1-1 Alberta Transportation Standard Drawings for Sealed Deck Joints

Alberta Transportation Standard Drawing	Joint Type	Maximum Permissible Normal Movement	Maximum Permissible Shear Movement
S-1810-20 (Type I Strip Seal Deck Joint – Sheet 1)	Multi-cell strip seal	55 mm	13 mm
S-1811-20 (Type I Strip Seal Deck Joint – Sheet 2)			
S-1812-20 (Type I Strip Seal Deck Joint – Sheet 3)			
S-1800-20 (Cover Plated V-Seal Deck Join – Sheet 1)	Cover plated 102 mm V-Seal	30 mm	20 mm

Alberta Transportation Standard Drawing	Joint Type	Maximum Permissible Normal Movement	Maximum Permissible Shear Movement
S-1801-20 (Cover Plated V-Seal Deck Joint – Sheet 2)	Cover plated 125 mm V-seal	55 mm	25 mm
S-1802-20 (Cover Plated V-Seal Deck Joint – Sheet 3)	Cover plated 178 mm V-seal	90 mm	30 mm

4-3.20 BARRIERS

4-3.20.1 General

A. Concrete barriers must have crack control joints in accordance with the details provided in Alberta Transportations standard drawing S-1412-20 at a maximum spacing of 3 m. The crack control joints must be sealed against moisture ingress. Barrier posts, if required, must be centred between crack control joints.

B. Base plates and anchors for barrier posts must be grouted. A minimum 40 mm nominal thickness grout pad must be provided under base plates. The grout must sit in a grout pocket recessed 20 mm nominal into the surface of the Transportation Structure. The grout pocket must be at least 40 mm larger than the base plate around the perimeter. The height of the base plates must be adjustable until after the barrier alignment has been fixed.

4-3.20.2 LRV Barriers

A. Transportation Structures carrying LRV and with an elevation difference of 600 mm or more between grade and top of deck must be provided with guardrail to prevent derailment. If guardrail cannot be provided LRV barriers must be used to prevent derailment. The LRV barriers must be sufficient to prevent Light Rail Vehicles from falling off the Structure, including due to tipping.

B. An LRV barrier may be considered to be sufficient to prevent a LRV from falling off of a Structure provided it meets the following requirements:

1. HFDG Section 9.9.6 - “Derailment Forces”

2. The LRV barrier has sufficient height to prevent a LRV from tipping over it. The height of the centre of gravity of the Light Rail Vehicle above the top of rail must be based on the existing High Floor LRV centre of gravity except that the centre of gravity must not be taken as being less than 1.8 m above the top of rail. Unless otherwise specified Transportation Structure components located behind LRV barriers must be setback a minimum of 600 mm from the traffic face of the LRV barrier.

4-3.20.3 Pedestrian Barriers

A. Pedestrian railing pickets must be vertical and have a maximum clear spacing no greater than 100 mm.

4-3.20.4 Attachments Behind LRV Barriers

A. Unless otherwise specified, attachments such as sign supports, OCS pole supports, lamp post supports, and sign structure supports mounted on top of or behind LRV barriers must be setback a minimum of 600 mm from the traffic face of the barrier.

4-3.21 CURBS

A. Sidewalks or SUPs on Transportation Structures must have curbs along their outside edges which project at least 100 mm above the finished top of sidewalk or SUP and are a minimum of 150 mm wide.

4-3.22 TRACKWAY SLABS

A. Joints between at-grade Trackway slab segments must allow for expansion and contraction to occur across the joint while preventing differential settlement across the joint.

1. The joints must be filled with a compressible material and sealed to prevent water ingress into the joint.

2. Stainless steel dowels must intersect the joint.

3. Joints must be no greater than 20 mm wide at the time of construction.

B. An approach slab must be provided under a Trackway slab to provide a smooth transition between a structurally supported Trackway slab to a ground supported Trackway slab or tie and ballast track.

1. Approach slab must be in accordance with Section 4-3.13.1.G [*General*] of this Schedule.

4-3.23 POLE FOUNDATIONS

A. OCS pole foundations must be drilled, cast-in-place, steel reinforced concrete piles with anchor bolts to attach to the pole. Where pile foundations are not practicable, poles may be mounted on bridge decks or structurally reinforced slabs.

B. OCS and other poles supported on Transportation Structures must fail before their foundations and anchorages. The factored resistances of the foundations and anchorages as determined in accordance with CAN/CSA S6 must be a minimum of 120% of the elastic factored resistances of the poles they support as determined in accordance with CAN/CSA S6.

1. The factored resistance of an OCS or other pole may be assumed to be limited to the factored resistance of a weaker link placed between the pole and pole anchorage/foundation which limits the amount of force that can be transferred between the pole and pole foundation/anchorage to the factored resistance of the link.

4-3.24 DIRECT FIXATION SUPPORT

A. The Design Life of direct fixation supports (i.e., plinths), including the fatigue limit state of the shear mechanism between the plinth and deck or slab, must be the same as the Design Life of the Transportation Structure supporting the direct fixation support.

B. Track fasteners and anchorages must not be directly anchored into a concrete deck.

4-3.25 TEMPORARY GROUND ANCHORS

A. Temporary ground anchors are ground anchors with a Design Life that is less than the Design Life of the Transportation Structure or Building Structure of which the ground anchors are part of.

B. Temporary ground anchors must be de-stressed no later than by completion of the applicable Work Package.

C. Temporary ground anchors must not extend beyond the boundary of the City Lands.

4-3.26 STAY CABLES & HANGERS

4-3.26.1 Design

A. Stay cables and hangers must comply with the requirements of PTI DC45.1 with the following modifications:

1. In Sections 5.1.1 to 5.1.4 of PTI DC45.1, the dead loads, live loads, fatigue loads and dynamic load allowance must be based on CAN/CSA S6, and Section 4-3 [*Design Guidance/Requirements*] of this Schedule rather than AASHTO LRFD.

2. In Sections 5.3.1 and 5.3.2 of PTI DC45.1, the design limit states, load factors and load combinations must be based on CAN/CSA S6, and Section 4-3 [*Design Guidance/Requirements*] of this Schedule rather than AASHTO LRFD. However, the construction limit states of Section 5.3.2.1 of PTI DC45.1 must apply.

3. In Section 5.3.5 of PTI DC45.1, the FLS load factor (ɣ) and stress range (ΔF) must be based on CAN/CSA S6, and Section 4-3 [*Design Guidance/Requirements*] of this Schedule rather than AASHTO LRFD. However, the nominal fatigue resistance (ΔF)n must be determined in accordance with PTI DC45.1, Section 5.3.5. AADTSL must be the average daily number of Light Rail Vehicles on one Track based on the System operating at the Maximum Service Level.

4. In Section 5.7.2.4 of PTI DC45.1 the structural design of saddle components must be based on CAN/CSA S6 and Section 4-3 [*Design Guidance/Requirements*] of this Schedule rather than on AASHTO LRFD.

4-3.26.1.1 Stay Cables & Hangers

A. The stay cables and hangers must be designed and detailed so that each stay cable or hanger can be installed, tensioned, removed and replaced individually.

B. The Transportation Structure must be designed to be serviceable for LRV traffic with any two hangers damaged or removed.

C. Stay cables and hangers must be designed and detailed to prevent ice buildup on the cables from falling on the Transportation Structure.

D. For each Transportation Structure with stay cables or hangers designed for single strand stressing, a minimum of eight additional reference strands must be distributed throughout the stay cables or hangers for corrosion and Design Life monitoring purposes.

1. The reference strands must be installed, stressed and protected under identical conditions to the other stay cable strands.

2. The stay cables and hangers must be designed such that any reference strands that are removed do not have to be replaced.

D. Design notes must be provided to show that the cables will not have excessive dynamic response and will not require damping to prevent excessive stress levels, structural fatigue, or user discomfort.

4-3.26.1.2 Anchorages

A. Anchorages must be capable of anchoring the full ultimate tensile force of the stay cables.

B. Anchorages must include cable guide systems that prevent significant bending stresses due to angular deviations of the strands from extending to the anchorages. Cable guide systems must not impose transverse forces on the Transportation Structure ahead of the anchorages.

C. Anchorages must be designed and detailed so that each stay cable can be installed, tensioned, removed and replaced individually.

D. Anchorages must be designed and detailed so that a force adjustment of 2.5% can be made simultaneously in all of the stay cable strands without the use of shims.

E. Anchorages must provide for a minimum of five percent additional strands for contingency. The holes provided for the additional strands must be positively sealed to protect the stay cable interior from corrosion.

SECTION 4-4 – CONSTRUCTION SPECIFICATIONS

4-4.1 BACKFILL

4-4.1.1 General

A. This Section 4-4.1 [*Backfill*] sets out the requirements for construction of backfills associated with or forming part of a Structure, including minimum requirements for fill material, placement, compaction, and testing.

B. Supply, placement, compaction and testing of fill materials must conform to the requirements of the D&CS, unless stated otherwise in this Section 4-4.1 [*Backfill*].

4-4.1.2 Materials

A. Aggregate materials must conform to the requirements in Tables 2.1.1, 2.1.2, and 2.1.3 of the D&CS Section 2.1 – Aggregates.

B. In-situ fill material from excavations or imported fill from borrow sources must be reviewed and approved by the Designer for the intended use.

C. Fill materials must be unfrozen, and free from ice, rocks larger than 75 mm, cinders, ashes, organic matter, refuse, Hazardous Substances or other deleterious materials.

4-4.1.3 Stockpiling

A. Stockpile fill materials in designated areas. Protect fill materials from contamination.

B. Stockpile granular materials in a controlled manner to minimize segregation.

C. Provide temporary erosion and sedimentation control measures to prevent soil erosion and discharge of soil-bearing run-off water or airborne dust to water bodies and adjacent properties and walkways, in accordance with Applicable Law.

4-4.1.4 Base/Subgrade Preparation

A. Remove and dispose of all materials at the base of excavation / subgrade that are deemed unsuitable as specified in the D&CS Section 2.3 – Grading, including but not limited to peat, roots, stumps, topsoil, frozen soil, frost susceptible material, garbage, construction debris or any other material deemed unsuitable by the Field Review Monitor. The requirements specified in Section 15 [*Contamination, Hazardous Substances and Waste*] and Section 18 [*Soil Salvage*, *Stockpiles and Imported Fill*] of Schedule 10 [*Environmental Performance Requirements*] apply to conducting this work.

B. Proof Rolling of the base/subgrade must be conducted in accordance with the D&CS Section 4.3 – Proof Rolling, with the following amendments.

   1. There must not be any discernable rutting or deflection during proof roll. Rutting and deflection requirements must conform to the following requirements:

      a. Clay Material: rutting or deflection of finished surface must not exceed 100 mm.

      b. Aggregate Material: rutting or deflection of finished surface must not exceed 50 mm.

   2. Rutting and/or deflection in excess of limits above must be considered a failure and must require the base/subgrade to be reworked and compacted to meet density and performance requirements.

3. After remedial work is performed, additional proof rolls must be performed to ensure the material conforms to the limits above.

4-4.1.5 Fill Placement and Compaction

A. Required compaction densities must be specified as percentages of maximum dry densities obtained from standard proctor laboratory tests in accordance with ASTM D698 (Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort).

B. Prior to placing any fill, the excavation base must be scarified to a minimum depth of 150 mm and compacted to a minimum of 95 percent of the standard proctor maximum dry density, as determined by ASTM D698.

C. Place backfill material in uniform lifts not exceeding 150 mm compacted thickness up to the grades indicated on the final Design Data. Compact each lift before placing the subsequent lift.

D. Compact each lift to the minimum requirements specified in the D&CS, or to higher densities as specified by the Designer based on Good Industry Practice. Under no circumstances must the compacted field density be less than 98 percent of the standard proctor maximum dry density.

E. Backfill material must be compacted at moisture contents within ± 2 percent of the optimum moisture content, as determined from standard proctor laboratory tests in accordance with ASTM D698.

F. Backfilling is not permitted when the average air temperature is expected to be below 0°C. The air temperature must be the temperature reported by Environment Canada. No backfill material will be permitted to be placed on frozen substrate.

G. The attained degree of compaction must be verified via field density tests at a minimum frequency of one test for each 1000 m^2 of compacted lift, with a minimum of one test per lift per day. Field density tests must be conducted in accordance with the requirements of ASTM D2922 test method.

H. A minimum of one laboratory standard proctor test (ASTM D698) must be conducted per 500 m^3 of backfill material to be placed on-site, per crushed aggregate source, or per non-granular material soil type, whichever is more frequent.

4-4.2 HEAVY ROCK RIPRAP

4-4.2.1 General

A. This Section 4-4.2 [*Heavy Rock Riprap*] sets out the requirements for the supply, delivery, and installation of riprap associated the Blackmud Creek LRT Bridge.

B. Supply, deliver and installation of heavy rock riprap must conform to the requirements of the D&CS, unless stated otherwise in this Section 4-4.2 [*Heavy Rock Riprap*].

4-4.2.2 Rock Material

A. Rock material must conform to the requirements in the D&CS with the exception of Table 4-4.2.2 [*Gradation Requirements*]. Gradation Requirements must instead follow the requirements of Table 4-4.2.2 [*Gradation Requirements*], in this Section.

Table 4-4.2.2 Gradation Requirements

Gradation		Heavy Rock Riprap Class			
Required Properties	**Units**	**1M**	**1**	**2**	**3**
Nominal Mass Nominal Diameter	kg	7	40	200	700
	mm	175	300	500	800
None greater than	kg	40	130	700	1800
	mm	300	450	800	1100
20% to 50%	kg	10	70	300	1100
	mm	200	350	600	900
50% to 80%	kg	7	40	200	700
	mm	175	300	500	800
100% greater than	kg	3	10	40	200
	mm	125	200	300	500

4-4.3 PILING

4-4.3.1 General

A. This Section 4-4.3 [*Piling*] sets out the requirements for steel H-piles, steel pipe piles, and cast-in-place concrete piles forming part of a Structure and OCS foundations, including minimum requirements for quality, supply, placement, curing, and testing of the piles.

4-4.3.2 Materials

4-4.3.2.1 Steel

A. Mill certificates for steel piles must be obtained prior to pile installation and must meet the requirements in Section 4-4.11.3.4 [*Mill Certificates*] of this Schedule.

4-4.3.2.2 Concrete

A. Pile Concrete must comply with the requirements of Section 4-4.6 [*Cast-In-Place Concrete*] of this Schedule.

4-4.3.2.3 Concrete Reinforcement

A. Concrete reinforcement must comply with the requirements of Section 4-4.10 [*Concrete Reinforcement*] of this Schedule.

4-4.3.3 Galvanizing

A. Galvanizing of steel piling, when required in the final Design Data, must be by the hot-dip method, in accordance with ASTM A123/A123M Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products.

B. Galvanized piling on which the galvanized coating has been damaged must be replaced or repaired in accordance with ASTM A780, Method A3 to a minimum thickness of 180 μm and tested for adhesion.

C. Where the upper portions of piling are specified to be galvanized on the applicable final Design Data, excess piling must be removed from the ungalvanized portion of the piling to ensure that the galvanized portion extends down to the elevation shown on the final Design Data.

4-4.3.4 Handling

A. Piling must be handled, hauled and stored in a manner that avoids damage to the piling materials.

B. Piling must be handled, hauled and stored in a manner that avoids damage to the galvanized surfaces on galvanized piling.

   1. Fabric slings, wood blocking or other methods must be used to support and separate galvanized piling when handling, hauling or storing.

4-4.3.5 Driven Steel Piles

4-4.3.5.1 Equipment

A. Pile driving equipment must be sized such that piles can be driven to the specified load capacity and required elevations without damaging the piles.

   1. The adequacy of the pile driving equipment must be based on wave equation analysis and/or PDA testing.

   2. Acceptable pile driving equipment includes diesel hammers, hydraulic hammers, vibratory hammers, and driving frames.

   3. The total energy developed by the hammer must not be less than 35 kJ per blow.

   4. Drop hammers must not be used.

   5. Pile driving equipment must use fixed leads. The use of hanging leads is not permitted.

B. The driving of piles with driving extensions must be avoided if practicable.

   1. When driving extensions are used, one pile from each group of ten must be a long pile driven without extensions and must be used as a test pile to determine the average bearing capacity of the group.

4-4.3.5.2 Tolerances

A. Piles must be placed in the positions, orientations and alignments shown on the applicable final Design Data.

   1. Precautions must be taken to ensure that the piles are in proper alignment, including the use of installation driving frames, fixed leads or other means.

B. Piles must not be out of the horizontal positions shown on the applicable final Design Data by more than 100 mm after driving except as noted below.

   1. For fully integral abutments the piles must not be out of horizontal position by more than 50 mm.

   2. For fully integral abutments, the variation in position between a pile casing centre and a pile centre must not be more than 25 mm.

C. Piles must not be driven with a variation of more than 20 mm per m from the vertical or from the batter shown on the applicable final Design Data.

D. Piles in exposed bents must not be out of position by more than 50 mm at the ground line or 25 mm in the pier cap.

E. At the completion of each driven pile within a foundation element, a control elevation must be established on the pile to determine if heave has occurred after all piles for the foundation element have been driven.

1. Piles that heave must be re-driven to the depth and capacity required.

4-4.3.5.3 Pile Driving Plan

A. A pile driving plan must be submitted to the City prior to commencement of pile installation work. The pile driving plan must include, as a minimum, the following items:

1. Specifications, setup and configuration of pile driving equipment, including:

   a. hammer data, including the hammer type, manufacturer, model number, serial number, maximum rated energy and range of operating energy, stroke at maximum rated energy, range of operating stroke, ram weight and modifications;

   b. details of onboard equipment capable of energy monitoring;

   c. striker plate data, including the weight, diameter, thickness and composition;

   d. hammer cushion data, including the manufacturer, area thickness per plate, number of plates, total thickness and composition;

   e. helmet data, including weight and composition; and

   f. pile cushion data, including material, area, thickness per sheet, number of sheets and total thickness of cushion.

2. Driving methods, procedure and driving sequence.

3. Details and drawings of driving frames.

4. Pile driving tools and accessories.

5. Mill test reports.

6. Proposed pile number convention.

7. Material tracking system sample output.

8. Pile lengths, splicing details, and anticipated splicing locations.

9. List of welders and proof of certification.

10. Welding procedures.

11. Environmental plan including a care of water plan to manage seepage water encountered during the driving of open ended pipe piles.

12. Cold weather protection methods.

13. PDA testing procedures, contact information, and qualifications of independent testing agency.

B. If during the course of Construction, the required pile set criteria and tip elevations are not achieved, a revised pile driving plan must be submitted to the City and Designer before any further pile driving continues.

4-4.3.5.4 Pile Driving

A. Steel driving frames must be used for driven pile installations.

B. Steel piles supplied with driving heads out of square must be cut square and neat using a cutting guide prior to commencement of the Project Work.

C. Sections of steel piles with holes cut into piling sections as part of the Design-Builder's rigging and lifting procedures must be cut at an elevation such that the holes are not incorporated into the Project Work or sized and installed in a location acceptable to the Designer. Plug welding of holes will not be permitted.

D. Where required by geotechnical conditions, pile tip reinforcement must be installed in accordance with Alberta Transportation Standard Drawing S-1850, Standard Steel Pile Details.

E. When pipe piles are to be driven closed ended, pipe pile tips must have welded end plates installed in accordance with Alberta Transportation Standard Drawing S-1850, Standard Steel Pile Details.

F. Piles must be driven to the tip elevations and the load capacities specified on the applicable final Design Data.

    1. Where practical driving refusal is encountered, the pile(s) may be terminated at a shallower elevation determined by the Designer that achieves the required pile stability and the specified minimum axial and lateral load capacities.

    2. If the required capacity at the design tip elevation is not achieved, re-strike tests may be performed after an appropriate waiting time of at least one week to confirm the pile capacity.

        a. Alternatively, the pile may be driven further to a deeper elevation and then assessed again to confirm that the required capacity is achieved.

G. The pile driving resistance (number of blows per 250 mm of pile penetration) at the termination of driving must meet or exceed the driving criteria specified by the Designer for the given pile section and length, piling equipment, soil conditions, and required pile load capacity.

    1. The driving termination criteria must be determined using wave equation analyses and verified using PDA testing.

H. Access to all foundation piles must be maintained until all PDA tests have been completed.

I. The pile head must be cut square and a driving cap or follower provided to hold the axis of the pile in line with the axis of the hammer.

    1. The follower must be of adequate dimensions to allow driving of the pile without trimming or reducing the cross section of the pile.

J. When damage or buckling is evident at the driving end of the pile before obtaining the required pile capacity or penetration of the pile, the driving end of the piling must be reinforced, or, other suitable equipment or procedures provided, to prevent further damage.

K. Piles must be cut off level at the required elevations shown on the applicable final Design Data.

L. At the completion of driving open ended pipe piles the thickness of accumulated material in the pipe pile must be measured and recorded by the Design-Builder. Upon completion of the measurements and review and acceptance of the Designer, the interiors must be cleaned out to the specified elevation.

M. All loose material and all material adhering to the inside walls of the piles must be removed to the satisfaction of the Designer prior to installing reinforcing steel or placing concrete.

N. Open pipe piles must be covered until concrete is placed.

O. Driving of any type of piles within 3 m of a pipe pile that contains concrete must not be undertaken until the concrete has been placed and cured for a minimum of 3 days.

4-4.3.5.5 Steel Pile Splices

A. Full strength pile splices must be used unless otherwise specified on the applicable final Design Data. Steel welding must be in accordance with Section 4-4.11.5 [*Welding*] in this Schedule.

B. Steel pile splices must be in accordance with Alberta Transportation Standard Drawing S-1850, Standard Steel Pile Details.

C. When splicing steel pipe piles, whatever means necessary must be employed to match out-of-round piling.

D. If splicing within the galvanized portion of a steel pile becomes necessary, the damaged galvanized area must be repaired in accordance with Section 4-4.11.8.2 [*Galvanizing*] of this Schedule.

E. Ultrasonic testing must be performed on a minimum of 20% of all pile splice welds except as noted below.

   1. Ultrasonic testing must be carried out on all tension splice welds as indicated on the applicable final Design Data.

   2. Ultrasonic testing must be carried out on welds where visual inspection indicates a potential Deficiency.

F. Ultrasonic testing must be carried out by a company certified to CAN/CSA W178.1.

   1. Ultrasonic testing technicians must be certified to Level II by the Canadian General Standards Board.

G. Welds must be repaired if full strength welds have not been achieved. Ultrasonic testing must be carried out on the repaired welds.

4-4.3.5.6 Defective Piles

A. A pile damaged by driving or driven out of proper location must be considered a Deficiency, which must be corrected by one of the following methods:

   1. The pile must be withdrawn and replaced by a new pile; or

   2. A replacement pile must be driven adjacent to the deficient pile. The top of the deficient pile must be cut to a minimum of 300 mm below the design cut off elevation.

B. All piles, pushed up by the driving of adjacent piles or by any other cause, must be driven down again to at least the original tip elevation.

4-4.3.6 Drilled Cast-in-place Concrete Piles

4-4.3.6.1 Pile Drilling Plan

A. The Design-Builder must have demonstrated experience related to the successful completion of previous projects with similar scopes of work and ensure that the equipment used is adequate in quantity, quality and operating condition.

B. A pile drilling plan must be submitted by the Design-Builder to the Designer for review and acceptance a minimum of 3 weeks prior to the commencement of pile installation. At a minimum, the following information must be provided in the pile drilling plan:

1. summary of Design-Builder's experience related to the project scope of work:
    a. Project personnel resumes
    b. Client contact information
2. specifications, setup and configuration of piling equipment;
3. drilling methods, installation sequence and timing, drilling procedure (including methodology to address the presence of boulders, hard rock stringers, or occurrences of pile wall sloughing and/or ingress of water);
4. list of drilling tools and attachments for belling and cleaning;
5. list of proposed casing lengths and diameters to be available on-site;
6. proposed use of drilling fluids and associated drilling fluid management considerations;
7. method of gas detection;
8. methods to install and remove temporary casing;
9. availability of borehole video inspection equipment and associated specifications;
10. concrete mix design review letter and applicable trial batch information in accordance with Section 4-4.6 [*Cast-In-Place Concrete*];
11. reinforcing steel mill test reports in accordance with Section 4-4.10 [*Concrete Reinforcement*];
12. reinforcing steel cage lift design drawings;
13. Tremie method plan, if applicable;
14. CSL testing details; CSL independent testing agency qualifications; and CSL tube grout mixing, placing and testing procedures, if applicable;
15. cold weather protection methods;
16. ECO Plan including a care of water plan to manage seepage water encountered; and
17. sample piling monitoring records.

C. If during the course of the Project Work, the required pile capacities and tip elevations are not achieved, or the Project Work is not being completed in accordance with the Project Requirements, the Design-Builder must revise his pile drilling plan and resubmit it to the City and Designer for review and acceptance before any further piling work occurs.

4-4.3.6.2 Drilling Pile Holes

A. Rotary type equipment is permitted for use Project wide. Hydraulic oscillating equipment will be permitted for the installation of temporary or permanent casings. Continuous flight auger equipment is not permitted for use at the Anthony Henday Drive LRT Bridge site.

B. Drilled pile holes must be stabilized and sealed by means of temporary casings or other methods to prevent possible collapse of the pile holes or ingress of water where sloughing and/or seepage conditions are expected or observed.

C. The drilling of pile holes must not proceed if adjacent piles may be damaged due to the effects of vibration or other reasons.

D. Every attempt necessary must be made to obtain "dry" pile holes prior to placing concrete.

   1. A pile hole must be considered “dry” if the seepage rate into the drilled pile hole is less than 25 mm in 5 minutes, and the water can be controlled by pumps or other means to reduce the depth of water at the bottom of the hole to 25 mm or less.

   2. All equipment necessary to achieve a dry hole must be available on-site and ready for use, including casings of appropriate size and length, bailing buckets, final cleanout buckets and water pumps.

   3. If a “dry” pile hole cannot be achieved, the pile must be filled with concrete by tremie methods.

E. Removal of temporary casing must not damage the Pile Concrete.

   1. Temporary casing, if used in drilling operations, must be removed from the hole as the Pile Concrete is being placed and before initial set of the concrete.

   2. The bottom of the casing must be maintained below the top of the concrete during casing withdrawal and concrete placing operations.

   3. Separation of the concrete during casing withdrawal must be avoided. Hammering or vibrating the casing to facilitate withdrawal operations is not permitted.

F. Drilled pile holes must extend to the elevations shown on the applicable final Design Data.

   1. Pile reinforcement and Pile Concrete must not be placed until the pile hole is deemed acceptable by the Field Review Monitor.

   2. The presence of any gas must be determined and appropriate means and equipment must be employed to ensure a safe work site.

4-4.3.6.3 Inspection and Testing

A. Where the end bearing resistance is relied upon in the pile design, the bottom of the excavated pile hole must be inspected with a down the hole video camera equipped with visual measurement scale to determine the depth of any loose sediments at the base.

   1. The camera must be capable of capturing a still image or record a video at a minimum resolution of 1080 p.

   2. The walls and bottoms of the pile holes must be cleaned to remove all loose and extraneous material.

   3. The bottom of the pile must be clearly visible to determine the cleanliness of the hole.

B. The inspection device/camera must be capable of verifying the cleanliness of the base in all expected piling conditions, including under water or drilling fluid.

C. For piles installed using continuous flight auger methods, Construction records measuring auger advance rate, auger withdrawal rate, concrete pumping rate and pressure, and comparison of theoretical hole volume and placed concrete volume must be documented for the entirety of the pile length.

D. For friction piles where the end bearing resistance is not considered in the pile design, the cleanliness of the base should be verified using weighted tape measurements.

4-4.3.6.4 Tolerances

A. Piles must be placed in the positions and alignments shown on the applicable final Design Data.

B. Piles must not be out of the horizontal positions shown on the applicable final Design Data by more than 50 mm.

C. Piles must not have a variation of more than 20 mm per m from the vertical or from the batter.

4-4.3.6.5 Concrete Reinforcement Placement

A. Concrete reinforcement projecting from a pile must be placed to a tolerance not exceeding 10 mm in any direction.

   1. Adequate "shoes" or spacers must be firmly anchored to the concrete reinforcement to ensure the concrete reinforcement is kept centred in the concrete when placing Pile Concrete.

B. Piles installed using continuous flight augers require the following provisions during concrete reinforcement placement:

   1. Reinforcement must be installed to the design depth immediately following concrete placement.

   2. The Design-Builder must provide means to keep the reinforcement centered within the concrete filled pile excavation to prevent the reinforcement from contacting the excavation side walls while lowering into the concrete filled pile excavation.

   3. The concrete reinforcement must be lowered into the concrete filled pile excavation by gravity or, if necessary, downward pressure by ground personnel. Hammering, mechanical driving, or vibrating of the concrete reinforcement is not permitted.

4-4.3.6.6 Concrete Placement

A. Under no circumstances must the pile holes be left open for periods longer than 2 hours.

B. If delay in placement of concrete reinforcement and concrete is expected, the pile hole must be properly backfilled and then redrilled when ready. The redrilled hole may be larger and/or deeper than the original pile hole, as determined by the Designer.

C. Forms must be used to maintain the specified dimensions of the portions of concrete piles above ground level.

D. If a dry pile hole cannot be achieved in accordance with the requirements of Section 4-4.3.6 [*Drilled Cast-in-place Concrete Piles*] of this Schedule, Pile Concrete must be placed in accordance with the requirements of Section 4-4.6.13.3 [*Concrete Placed Under Water*] of this Schedule.

1. Pile Concrete placed under water must be validated by CSL in accordance with Section 4-4.6.13.3 [*Concrete Placed Under Water*] of this Schedule*.*

E. The concrete below ground specified in this section is exempt from the requirements of Section 4-4.6.18 [*Concreting in Cold Weather*] of this Schedule*.*

F. When the temperature of the ground against which Pile Concrete is placed is below 0ºC the concrete must be protected from heat loss as follows.

1. The pile hole diameter must be oversized by 100 mm to at least below the design frost depth.

2. Immediately after placing and finishing the concrete, the top exposed surface of the pile must be protected with insulated tarps or other means to adequately cure the concrete in accordance with Section 4-4.6.20 [*Curing Concrete*] of this Schedule for a minimum period of 7 days.

3. If the top of the pile extends above the ground surface it must be protected in accordance with Section 4-4.6.18 [*Concreting in Cold Weather*] of this Schedule.

4. Raising the temperature of concrete reinforcement which concrete will be placed against to a temperature of between 5 and 20ºC before placing any concrete.

4-4.3.6.7 Defective Piles

A. Manipulation of reinforcement to adjust position, considered by the Designer to have the potential to damage the pile integrity, will not be permitted.

B. A decrease in concrete elevation at the top of a completed pile of more than 2% of the pile diameter between the time of initial pour and set will be considered a damaged pile.

C. Piles damaged during drilling, reinforcing steel cage installation, or concrete placement must be repaired by the Design-Builder at his expense. The Design-Builder must submit a repair procedure to the Designer for review and acceptance before any further piling is installed. The Design-Builder’s repair procedures, as a minimum, must include one of the following:

1. the defective pile(s) must be redrilled and replaced by new, and if necessary, longer piles;

2. concrete removal and replacement;

3. the defective piles must be built up; or

4. the pile cap extended and reinforced to the satisfaction of the Designer.

4-4.3.7 Pile Capacity Testing

4-4.3.7.1 Static Load Testing

A. Static load tests may be used for the determination of pile capacity.

1. Static compression load tests must comply with ASTM D1143/D1143M. Uplift static load tests must comply with ASTM D3689/D3689M.

B. Osterberg or Statnamic load tests may be used in place of static load tests.

1. Osterberg tests must conform to ASTM D1143 Standard Test Method for Piles under Static Axial Load using the Quick Load Test Method for Individual Piles.

2. Statnamic tests must conform to ASTM D7383 Standard Test Methods for Axial Compressive Force Pulse (Rapid) Testing of Deep Foundations.

C. If higher geotechnical resistance factors are to be used in the design of a Structure, they must be justified as follows:

1. a minimum of one pile load test must be performed for each Structure;

2. for Structures longer than 500 m, a minimum of one pile load test must be performed for each 500 m long segment of the Structure; and

3. the frequency of testing must be increased as necessary to account for changing soil conditions, pile sections and types and construction methods.

4-4.3.7.2 Pile Driving Analyzer (PDA) Testing

A. PDA testing may be used for the determination of pile capacity.

1. PDA testing must comply with ASTM D4945.

2. A minimum of two accelerometers must be installed on each tested driven pile and four accelerometers on each tested cast-in-place concrete pile.

3. All accelerometers and transducers must be calibrated and inspected to ensure proper attachment to the pile.

4. The impact imparted on the pile must be sufficient to fully mobilize the skin friction and end bearing resistances of the pile and must result in a net permanent set per blow of between 3 mm and 8 mm upon impact from the pile hammer.

5. The hammer energy used during PDA testing must be such that the required ultimate pile capacity is mobilized in a single blow without additional data interpretation.

6. Under no circumstances must the pile capacity be based on the superposition of toe and shaft resistances from different strikes, re-strikes or any combination thereof.

B. If PDA testing is required by the final Design Data, the greater of two piles or 15% of the piles at each Structure must be tested, including tests at each substructure element associated with the Structure and at each different soil condition encountered.

1. The piles selected for PDA testing must be representative of other piles in the same Structure.

2. Where driven piles exhibit lower driving resistances or shorter penetrations than normal, or where cast-in-place concrete piles experience extraneous soil, groundwater, and/or installation conditions, additional PDA tests over and above the minimum number of tests specified above must be required.

3. Additional PDA tests must accompany changes in piling equipment, piling procedures and pile requirements.

C. For driven piles, PDA testing must be conducted at the end of initial driving and upon re-strike when re-striking is required.

1. Where time dependant changes in the soil conditions are anticipated, such as pile setup or relaxation, additional re-strike PDA tests must be conducted on a sample of previously tested piles to determine the bearing parameters after driving induced pore pressures have dissipated.

2. The re-strike PDA tests must be conducted no sooner than one week after initial driving, or longer as directed by the Designer.

3. Where the capacity of the pile at re-strike is relied upon for design, a minimum of one half of the piles PDA tested during initial driving must be PDA tested again during re-strike.

D. For cast-in-place concrete piles, the PDA testing must be conducted no sooner than one week after the installation of the pile.

E. If one pile in a pile group does not meet capacity requirements, additional testing must be carried out to confirm if the pile is an isolated case.

F. The PDA testing agency must prepare a field report summarizing the preliminary PDA testing results, including driving stresses, transferred energy and estimated pile capacity, within 24 hours of testing, and such report must be submitted to the City promptly thereafter.

G. The PDA testing agency must complete a final testing report, complying with ASTM D4945, within 7 Business Days of PDA testing, and such report must be submitted to the City promptly thereafter. As a minimum, the report must include the following:

1. pile and driving system information;
2. pile installation data;
3. PDA testing equipment and procedure;
4. energy imparted to the piles;
5. maximum driving stresses;
6. hammer blow rate;
7. CAPWAP input parameters including quake and damping factors; and
8. shaft friction, end bearing and total pile capacity.

H. The PDA test results must be used to confirm/update the pile driving termination criteria and determine the requirements for modification of pile driving procedures or equipment, and for pile acceptance.

4-4.3.8 Pile Integrity Testing

A. For piles installed using continuous flight auger methods, assessment of the integrity of piles using a low strain non-destructive pile integrity tester or equivalent must be undertaken on a minimum of 50% of the piles at each structure, including tests at each substructure element associated with the structure and at each different soil condition encountered. Reports must be submitted to the City.

B. Where integrity testing indicates potential shaft or pile defects such as major cracks, necking, bulging, soil inclusions, or voids the respective pile(s) will be considered defective.

4-4.4 MICROPILES

4-4.4.1 General

A. This Section 4-4.4 [*Micropiles*] sets out the Construction requirements for micropiles forming part of a Structure, including minimum requirements for supply, installation, grouting and testing.

4-4.4.2 Engineering Data

4-4.4.2.1 Related Design-Build Construction Requirements

A. Section 4-4.6 [*Cast-In-Place Concrete*] of this Schedule.

B. Section 4-4.10 [*Concrete Reinforcement*] of this Schedule.

C. Section 4-4.11 [*Structural Steel*] of this Schedule.

4-4.4.2.2 Shop Drawings

A. Shop drawings showing fabrication and installation details of the micropiles must be submitted to the City. The shop drawings must include the following:

   1. batter, length, and diameter of micropile;

   2. type, length and size of the micropile steel bar;

   3. type, length, diameter, wall thickness, and elevation of permanent steel casing (if applicable);

   4. required ultimate geotechnical resistance of micropile;

   5. anchorage connection details to the Structure footing(s);

   6. corrosion protection system for the reinforcing bar, couplers and anchorage components;

   7. type and spacing of reinforcing bar centralizers and spacers; and

   8. grout mix design and grout placement procedures, including post-grouting details (if applicable).

4-4.4.2.3 Mill Certificates

A. Mill certificates for the micropile steel components (e.g., steel bars, couplers, casings, plates), including the ultimate strength, yield strength, load/elongation curves, and composition, must be provided to the City.

B. Manufacturer certificates of compliance for the micropile centralizers and spacers must be provided to the City.

C. Mill test reports originating from a mill outside of Canada or the United States of America must meet the requirements of Section 4-4.11.3.4 [*Mill Certificates*] of this Schedule.

4-4.4.3 Materials

A. Materials for micropiles must comply with Section 4-3.5.9 [*Micropiles*] and Section 4-3.6.4.5 [*Micropile Corrosion Protection*] and FHWA-NHI-05-039.

B. Reinforcing bar encapsulation (for double corrosion protection) must be shop fabricated using high-density, corrugated polyethylene tubing conforming to the requirements of ASTM D3350 with a nominal wall thickness of 0.8 mm. The inside annulus between the reinforcing bar and the encapsulating tube must be a minimum of 5 mm and be fully grouted with non-shrink grout.

C. Steel bar couplers must develop 120 percent of the specified tensile yield strength of the bar as certified by the manufacturer.

D. Centralizers and spacers must be fabricated from Schedule 40 PVC pipe or tube, steel, or material non-detrimental to the steel bar. The use of wood is not permitted.

E. Permanent steel casing, if used, must conform to the requirements of API, Grade N80.

F. Micropile grout must be neat cement or sand/cement mixture with a minimum 3 day compressive strength of 21 MPa and a minimum 28-day compressive strength of 35 MPa in accordance with ASTM C109.

G. Admixtures, if used, must meet the requirements of ASTM C494, and must be compatible with the grout. The use of accelerators is not permitted. Expansive admixtures may only be used for filling anchorage covers.

4-4.4.4 Installation

4-4.4.4.1 General

A. The entity performing any micropiling must be experienced in the construction and load testing of micropiles and have successfully constructed at least five projects in the last 5 years involving construction totaling at least 500 micropiles of similar capacity to those required in the final Design Data.

B. Micropile materials, including steel bars, anchorage components, cement and admixtures, must be handled, stored and installed in such a manner as to avoid damage, corrosion or contamination with dirt or deleterious substances, in accordance with the requirements of FHWA-NHI-05-039.

C. The use of drilling fluids (such as bentonite slurry) to advance micropile holes is not permitted.

4-4.4.4.2 Installation Tolerances

A. Micropiles must be installed at the locations, elevations, and orientations shown on the applicable final Design Data.

B. The centres of micropiles must not deviate from the horizontal locations shown on the applicable final Design Data by more than 50 mm in any direction.

C. Micropiles must not deviate by more than 15 mm per m from the vertical or from the batter shown on the applicable final Design Data.

D. Micropile reinforcing bars must be placed within 20 mm of the centre of the drilled hole.

E. The cut off elevations of micropiles must be within plus or minus 25 mm of the elevations shown on the applicable final Design Data.

F. Micropiles that do not satisfy the specified tolerances must be replaced.

4-4.4.4.3 Drilling

A. The micropile drill hole must be open along its full length prior to placing the reinforcing bar and the grout. Where the reinforcing bar cannot be installed to the design depth easily, the micropile hole must be redrilled and adequately supported to facilitate bar insertion.

4-4.4.4.4 Grouting

A. Grouting of micropiles must comply with FHWA-NHI-05-039.

B. Grouting of the drill hole after the installation of the steel bar must be completed within 2 hours of completion of drilling and must be done in one continuous operation. Cold joints in the grout column are not permitted.

C. The grout must be free of lumps and undispersed cement.

D. Admixtures, if used, must be mixed in accordance with the manufacturer's recommendations.

E. The compressive strength of micropile grout must be tested in accordance with ASTM C109 at a frequency of no less than one test for every 5 m^3 of grout placed. Irrespective of the volume of grout placed, a minimum of one test must be performed on a set of grout cubes from each grout plant on each day of operation.

F. The grouting equipment must be sized to enable the grout to be pumped in one continuous operation.

1. The mixer must be capable of continuously agitating the grout.

2. The grout must be placed within one hour of mixing.

G. A positive displacement grout pump must be used.

1. The pump must be equipped with a pressure gauge to monitor grout pressure. A second pressure gauge must be placed at the point of injection at the micropile top.

2. The pressure gauges must be capable of measuring pressures of a least 1.0 MPa or twice the actual grout pressures required, whichever is greater.

4-4.4.4.5 Installation Records

A. An installation record must be prepared for each micropile and must include:

1. a unique reference number for each micropile;

2. the date of installation and weather conditions;

3. soil and groundwater conditions encountered during drilling;

4. the As-Built location, and batter of micropile;

5. final cut off and tip elevations;

6. the As-Built length and diameter of micropile;

7. grade, size, and length of the steel bar;

8. top and tip elevations, grade, diameter, wall thickness, and length of permanent steel casing (if applicable); and

9. quantity of grout, grout pressure and, if applicable, details of post-grouting stages (number of stages, date/time of each stage, grout volume and pressure, etc.).

4-4.4.5 Load Testing

A. The load carrying capacity and overall performance of micropiles, including creep behavior, must be evaluated for acceptance using verification tests on sacrificial pre-production micropiles and proof tests on production micropiles, in accordance with the recommendations of FHWA-NHI-05-039.

B. A minimum of one verification test must be performed in each anticipated major soil/rock strata at the location of each Structure. For lineal structures, a minimum of one verification test must be performed for each 500 m long segment of the Structure. The diameter, embedment depth, and equipment and installation procedures for the verification test micropiles must be identical to those specified for the production micropiles. The test setup, testing procedures and results of verification tests must be reviewed and accepted by the Designer prior to the installation of production micropiles. Verification test micropiles must be sacrificial and must not be incorporated as production micropiles.

C. Proof tests must be carried out on a minimum of 5% of the production micropiles, with at least one proof test per each Structure foundation. The frequency of proof tests must be increased where variable subsurface soil conditions are anticipated or where proof tests on production micropiles in the same structure failed to meet the test acceptance criteria.

D. Verification test micropiles must be loaded incrementally in accordance with the loading schedule in Table 7.1 of FHWA-NHI-05-039. Proof test micropiles must be loaded incrementally in accordance with the loading schedule in Table 7.2 of FHWA-NHI-05-039. The micropile movements must be recorded at each load increment. For the purposes of testing, the “Design Load” in Tables 7.1 and 7.2 of FHWA-NHI-05-039 must be taken as the governing SLS load detailed on the applicable final Design Data.

E. The travel of testing equipment (e.g., jack ram, dial gauges, transducers, etc.) must be sufficient to allow the test to be completed without resetting the equipment.

F. Measurements of applied loads during verification and proof tests must be recorded using load cell and jack pressure gauge calibrated within 6 months prior to testing. The load cell must be properly aligned with the axis of the micropile and jack.

G. Acceptance criteria for verification and proof tests must be in accordance with the requirements of FHWA-NHI-05-039. The following additional criteria must also be met for both tests:

   1. The axial movement at the top of the test micropile when subjected to compression or tension load equal to 1.6 times the “Design Load” must not exceed 25 mm

   2. The axial movement at the top of the test micropile at 1.0 times the “Design Load” must meet the design criteria for the Structure as determined by the Designer.

   3. The creep rate throughout the creep load hold period must be constant or decreasing.

H. Micropiles not meeting the acceptance criteria must necessitate modification of the design, the construction procedure, or both. These modifications may include installing replacement micropiles, incorporating the micropiles at a reduced load capacity of not more than 50 percent of the maximum load attained during testing, post-grouting, modifying the installation methods, increasing the bond length, or changing the micropile type. Any modifications must require additional verification and proof testing.

4-4.5 GROUND ANCHORS

4-4.5.1 General

A. This Section 4-4.5 [*Ground Anchors*] sets out the requirements for all ground anchors and other permanent structural components resisting lateral earth load/surcharge load and forming part of a Structure, including minimum requirements for supply, installation, grouting and stressing.

4-4.5.2 Engineering Data

4-4.5.2.1 Shop Drawings

A. Shop drawings showing fabrication and installation details of the ground anchors must be submitted to the City. The shop drawings must include the following:

   1. the type and size of the ground anchor tendons;

2. the ground anchor design loads;
3. the minimum tendon bonded lengths, unbonded lengths, and total lengths;
4. tendon anchorage details, including details of any trumpets;
5. the corrosion protection system for the tendons and anchorages; and
6. the type and spacing of tendon centralizers and spacers.

4-4.5.2.2 Mill Certificates

A. Mill certificates for the tendons, including load/elongation curves, must be provided to the City.

B. Mill certificates for the anchorages must be provided to the City.

C. Mill test reports originating from a mill outside of Canada or the United States of America must meet the requirements of Section 4-4.11.3.4 [*Mill Certificates*] of this Schedule.

4-4.5.3 Materials

A. Material for ground anchors must comply with PTI DC35.1, and Section 4-3.15.4 [*Retaining Walls with Ground Anchors*] and Section 4-3.6.4.4 [*Ground Anchor Corrosion Protection*] of this Schedule.

B. Admixtures, if used, must meet the requirements of ASTM C494 and PTI M55.1, and must be compatible with the grout, any admixtures, and tendon components. The use of accelerators is not permitted. Expansive admixtures may only be used for filling sealed encapsulations, trumpets, anchorage covers.

4-4.5.4 Installation

4-4.5.4.1 General

A. Ground anchor tendons, including anchors and prestressing steel, must be handled, stored and installed in such a manner as to avoid damage, corrosion or contamination with dirt or deleterious substances.

B. Ground anchors must be handled, stored and installed in accordance with the requirements of PTI DC35.1.

C. Tendon tails must be cleaned and protected from damage until final testing and lock-off.

4-4.5.4.2 Installation Tolerances

A. Ground anchors must be placed to a horizontal tolerance of plus or minus 50 mm.

B. Ground anchors must not be out of slope, batter or alignment by more than 20 mm per metre.

C. The ground anchor anchorages must be installed perpendicular to the tendons, without bending or kinking of the tendons.

4-4.5.4.3 Grouting

A. Grouting of ground anchors must comply with PTI DC35.1.

B. The grout must be free of lumps and undispersed cement.

C. Admixtures, if used, must be mixed in accordance with the manufacturer’s recommendations.

D. The grouting equipment must be sized to enable the grout to be pumped in one continuous operation.

1. The mixer must be capable of continuously agitating the grout.

E. A positive displacement grout pump must be used.

1. The pump must be equipped with a pressure gauge to monitor grout pressure.

2. The pressure gauge must be capable of measuring pressures of a least 1.0 MPa or twice the actual grout pressures required, whichever is greater.

4-4.5.4.4 Load Tests

A. The load capacity and overall performance of ground anchors, including load-extension behavior, relaxation and creep, must be evaluated for acceptance using pre-production tests, performance tests and proof tests in accordance with the requirements of PTI DC35.1.

1. A minimum of one pre-production test must be performed in each anticipated major soil/rock strata at the location of each Structure. Pre-production tests must satisfy the minimum requirements of the performance test. The test setup, testing procedures and results of pre-production tests must be reviewed and accepted by the Designer prior to the installation of production ground anchors.

B. Performance tests must be carried out on a minimum of 5% of the production ground anchors to confirm the adequacy of the design, materials, and method of construction.

1. The performance tests must be carried out by cyclically and incrementally loading and unloading the ground anchor to a minimum test load of 1.33 times the design load in accordance with PTI DC35.1.

2. The performance tests must be carried out on ground anchors constructed under methods and conditions identical to those used on the Project.

3. The frequency of performance tests must be increased when subsurface conditions are variable and/or the ground anchor load capacities are in question.

C. Proof tests must be carried out on all production ground anchors not subjected to performance tests. The proof tests must be conducted by incrementally loading the anchor up to 1.33 times the design load in accordance with PTI DC35.1.

D. After load testing has been completed, the load in the tendons after seating losses must be within ± 5% of the specified lock-off load shown on the applicable final Design Data.

E. After transferring the load to the anchorage, and prior to removing the jack, a lift off test must be carried out to confirm the magnitude of the load in the ground anchor tendon.

1. This load is determined by reapplying load to the tendon to lift off the wedge plate without unseating the wedges.

4-4.5.5 Ground Anchor Installation Records

A. A ground anchor installation record must be provided to the City or each ground anchor installed and must include:

1. a unique reference number for each ground anchor;

2. the date of installation and weather conditions;

3. the Field Review Monitor’s name;

4. the As-Built location and orientation of each ground anchor;

5. pertinent information regarding the ground anchor installation including design load, installation procedure used, anchor type, completed overall anchor length, anchor embedment length, soils encountered during drilling, water table, casing (if used), anchor material(s) used, complete geometric information, stressing information (if stressed elements were used), grouting and post-grouting information, including quantity of grout and grout pressure used, and any difficulties encountered. The information must be suitable for a complete independent design load, resistance and durability assessment; and

6. documentation of the load test and load test results.

B. The ground anchor installation records must be authenticated by a Professional Engineer.

4-4.5.6 Basis for Rejection

A. A plan for resolution, demonstrating that the Project Work will confirm to the requirements of this schedule, must be submitted to the City in the event of any of the following circumstances:

1. Ground anchors being installed out of geometric tolerance.

2. Ground anchors not meeting the required load resistance or the design performance criteria, including residual movement and creep rate.

3. Ground anchors that encounter unforeseen or excessively variable subsurface conditions that detrimentally affect the load resistance or durability of the anchor.

4-4.6 CAST-IN-PLACE CONCRETE

4-4.6.1 General

A. This Section 4-4.6 [*Cast-In-Place Concrete*] sets out the requirements for all cast-in-place concrete, which includes cementitious products such as grout and concrete patching materials, forming part of a Structure, including minimum requirements for quality, sampling and testing of constituent materials of concrete, methods of producing and handling constituent materials, and batching, mixing, handling, transporting, placing, curing and finishing of cast-in-place concrete.

1. Additional requirements for cast-in-place concrete segmental construction are given in Section 4-4.9 [*Concrete Segmental Construction*] of this Schedule.

2. Concrete elements that are cast-in-place but not in its final location must meet the following additional requirements:

   a. Sections 4-4.7.9 [*Dimensional Tolerances*], 4-4.7.14 [*Handling and Storage*], 4-4.7.15 [*Erection of Precast Concrete Units*] of this Schedule.

   b. Must be fabricated in a production facility that is certified by the CPCQA Certification Program.

4-4.6.2 Materials for Cast-in-Place Concrete

A. All constituent materials for cast-in-place concrete must be selected to provide concrete with sufficient durability to meet the Design Life requirements of the Structure and sufficient strength to meet structural strength requirements.

B. Cast-in-place concrete must consist of hydraulic cement, water, aggregates, admixtures, silica fume, steel fibres and/or fly ash, as follows:

    1. Materials originating from outside Canada or the United States of America intended for use in the production of concrete must be tested to the required standard by a laboratory in Canada certified to CSA A283.

4-4.6.2.1 Portland Cement

A. Portland cement must comply with CAN/CSA A3001. General use (Normal), Type GU, must be used unless otherwise specified herein.

    1. Concrete intended for placement in sulphate environments may be produced with combinations of Type GU cement and supplementary cementing materials provided current CAN/CSA A3004-C8 test data demonstrates compliance with CAN/CSA A3001 requirements for high sulphate resistance.

B. For Building Structures Portland cement must comply with CAN/CSA A3001. General use (Normal), Type GU, or high sulphate resistant, Type HS or HSb cement must be used unless otherwise specified herein.

4-4.6.2.2 Water

A. Water for mixing concrete, concrete patching materials, concrete finishing materials or mortar must comply with CAN/CSA A23.1 and must be free from harmful amounts of alkali, organic materials and other deleterious substances.

    1. Slurry water, treated wash water or water from shallow, stagnant or marshy sources must not be used.

4-4.6.2.3 Aggregates

A. Fine and coarse aggregates must comply with CAN/CSA A23.1 and Section 4-4.6.4.3 [*Aggregate Tests*] of this Schedule.

4-4.6.2.4 Admixtures

A. Admixtures must meet the following requirements.

    1. Admixtures must be compatible with all mix constituents.

    2. Water reducing admixtures and superplastizing admixtures must comply with ASTM C494/C494M.

    3. Air entraining admixtures must comply with ASTM C260/C260M.

    4. Hydration stabilizing admixtures must comply with ASTM C494/C494M for Type B and/or Type D water reducing and retarding admixtures.

        a. Hydration stabilizing admixtures are only permitted as follows:

            i. when haul times are reasonably expected to exceed the times specified in Section 4-4.6.7 [*Time of Placing*] of this Schedule;

            ii. where hydration stabilization is required due to mass concrete placement considerations; and

iii. where the Design-Builder has prepared and submitted to the City a report justifying the use of such admixtures.

5. Calcium chloride, air reducing admixtures or accelerating admixtures are not permitted.

6. Anti-washout admixtures must conform to the US Army Corps of Engineers CRD-C 661.

B. Admixtures not noted above is not permitted without the prior written consent of the City, in its discretion.

4-4.6.2.5 Silica Fume

A. Silica fume must comply with CAN/CSA A3001 for a Type SF supplementary cementing material, with a minimum SiO_2 content of 85%, a maximum loss on ignition of 10% and maximum SO_3 content of 1%.

B. A compatible superplasticizing admixture must be used together with the silica fume.

4-4.6.2.6 Steel Fibres

A. Steel fibres must comply with ASTM A820/A820M, Type 1 or 5 and must be 50 mm in length with an aluminum content of no more than 0.020% by mass, when tested in accordance with test method Environmental Protection Agency 3050B.

B. Steel fibres must be free from balls and clumps at all times during their use.

C. Where the use of steel fibres is specified in the applicable final Design Data, Novocon XR or Wiremix W50 steel fibres at a dosage rate of 60 kg per m^3 of concrete must be used.

D. Alternative steel fibres and dosage rates may be used provided that their toughness (T^D_{600}) determined in accordance with ASTM C1609 is greater than or equal to that of the specified fibres and dosage.

E. Concrete with steel fibres is not permitted for Building Structures.

4-4.6.2.7 Fly Ash

A. Fly ash must comply with CAN/CSA A3001, for Type “F” fly ash with calcium oxide content (CaO) not exceeding 12%.

4-4.6.2.8 Grout

A. Grout and concrete patching materials must be packaged in waterproof containers with the production date and shelf life of the material shown.

4-4.6.3 Storage of Materials

A. All constituent materials for cast-in-place concrete must be stored separately in a manner that prevents contamination or deterioration.

1. All hydraulic cement, silica fume, fly ash and steel fibres must be stored in a manner that protects them from moisture.

2. All hydraulic cement, silica fume and fly ash must be free from lumps at all times during their use.

B. All aggregates must be stored and handled so as to prevent segregation, provide uniformity of materials and prevent contamination.

1. Separated aggregates, aggregates secured from different sources, and fine and coarse aggregates must be stored in separate stockpiles.

2. The sites of all stockpiles must be cleared of all foreign materials and must be level and firm.

3. If aggregates are placed directly on the ground, aggregates within 150 mm of the ground level must not be used and this material must remain undisturbed to avoid contaminating the aggregate being used.

4. Aggregates must be handled in a manner that prevents segregation.

4-4.6.4 Concrete Mix Design and Aggregate Tests

4-4.6.4.1 Concrete Mix Design

A. Design all concrete mixes to provide concrete that:

   1. is sufficiently workable, for the applicable placement and finishing requirements;

   2. has sufficient durability to meet the Design Life of the Structure; and

   3. has sufficient strength to meet structural strength requirements.

B. Submit to the City a concrete mix design review letter, together with applicable material quality compliance test reports, for each class of concrete before first placement of such concrete.

   1. The mix design review letters must include the following items:

      a. Evaluation and summary of all mix constituents

      b. Material test reports

      c. Mix proportion quantities

      d. Trial batch results (if applicable)

      e. Mass concrete design considerations (if applicable)

      f. Portable batch plant batching procedures (if applicable)

   2. Each concrete mix design review letter, including the applicable material test reports, must be authenticated by a Professional Engineer engaged by an independent concrete testing laboratory certified to CAN/CSA A283. The certifying Professional Engineer must also provide a professional opinion confirming that the concrete mix is suitable for the intended use and can be expected to meet all applicable Project Requirements over the Design Life of the Structure.

   3. Where concrete will be placed by concrete pump, the concrete mix must be specifically designed for pumping.

   4. Where concrete will be placed under water, the concrete mix must be specifically designed for placing concrete under water.

      a. An excess of 15% above the cement quantity required by the equivalent conventionally placed concrete mix design must be provided.

      b. The concrete mix may contain an “anti-washout” admixture incorporating viscosity modifiers (Whelan gum, etc.) to enhance the performance of the mix.

C. Any proposed modifications made to the concrete mix design must be submitted to the City in accordance with the requirements of this Section 4-4.6.4.1 [*Concrete Mix Design*].

D. Unless otherwise noted, submission of a concrete mix design review letter is not required for prebagged grouts and concrete patching materials provided they are mixed in strict accordance with the manufacturer's recommendations.

4-4.6.4.2 Mass Concrete

A. If a concrete element has been designated as mass concrete, the Design-Builder must develop and submit a mass concrete thermal control plan, including thermal mitigation measures, to the Designer for review and acceptance. The mass concrete thermal control plan must be submitted a minimum of 4 weeks prior to the commencement of mass concrete placement. The mass concrete thermal control plan will be considered a Professional Work Product and must be authenticated by a Professional Engineer licensed to practice in the Province of Alberta and validated by a Responsible Member, in accordance with APEGA requirements. The mass concrete thermal control plan must be in accordance with CSA A23.1 Subsection 7.6.3.4, Thermal Control Plan. Thermal control mitigation measures must be developed and consistent with the technical requirements outlined in the CSA A23.1 Annex T, Subsection T.5 Thermal Control Planning and the ACI 207.4R, Cooling and Insulating Systems for Mass Concrete. The mass concrete thermal control plan must also describe the performance monitoring requirements and on-site response strategies to address varying thermal conditions.

4-4.6.4.3 Aggregate Tests

A. A break in production of a particular class of concrete must not constitute the need for additional testing if the following items are submitted to the City:

1. Aggregate sieve analysis;
2. Organic impurities in sands for concrete;
3. Petrographic examination of aggregates; and
4. Letter of Evaluation prepared by the professional signatory of the mix design review letter indicating that the material initially tested is still representative.

B. Fine aggregate must be tested in accordance with CAN/CSA A23.2-7A, "Organic Impurities in Sands for Concrete".

1. Fine aggregate producing an organic impurity colour darker than the Standard colour must be rejected in the absence of a satisfactory record of performance in a similar class of concrete (minimum 30 tests over the last 12 months). Clauses 4.2.3.3.3.2 (a) and (b) of CAN/CSA A23.1-14 must not apply.
2. Testing in accordance with CSA A23.2-2A and 5A must not have more than 3.0% passing an 80 μm sieve.

C. The potential for deleterious alkali-aggregate reactivity for fine and coarse aggregates must be assessed in accordance with CAN/CSA A23.2-27A. This assessment must include the risk level associated with Structure size and environment, the level of prevention required to achieve the Design Life of the Structure and the determination of the appropriate preventative measures, including testing in accordance with CAN/CSA A23.2-28A. Current (less than 24 months old) test data evaluating the potential alkali-silica reactivity of aggregates tested in accordance with CAN/CSA A23.2-14A or CAN/CSA A23.2-25A is required. In the absence of test data, the aggregate must be considered highly reactive.

D. Petrographic analysis on the coarse and fine aggregates must be performed in accordance with CAN/CSA A23.2-15A by experienced personnel employed by a CAN/CSA A283 certified laboratory. The petrographic analysis report must be authenticated by either a Professional Engineer, a Professional Geologist, or a Geological Engineer.

  1. The (weighted) petrographic number of the coarse aggregates must not exceed 130, and the ironstone content must not exceed 0.8%.

  2. Ironstone content in fine aggregate (material retained on the 2.5 mm sieve) must not exceed 1.5% by total dry mass of fine aggregate for all classes of concrete in Table 4-3.5.1-1 [*Concrete Classes*] except Pile Concrete.

E. Material test reports must be current according to the required frequency of analysis in Table 4-4.6.4-1 [*Material Test Frequency*] and fully represent materials to be used in concrete production. For each mix design submission, the source(s) of aggregate(s) and following aggregate analysis must be provided to the City.

Table 4-4.6.4-1 Material Test Frequency

Aggregate Analysis	Standard	Required Frequency of Analysis (maximum days prior to production)
Fine and coarse aggregate sieve	CAN/CSA A23.2-2A	90
Amount of material finer than 80 µm in aggregate	CAN/CSA A23.2-5A	90
Organic impurities in sands for concrete	CAN/CSA A23.2-7A	90
Results of deleterious substances and physical properties of aggregates	Table 12, CAN/CSA A23.1; A23.2-3A, A23.2-4A, A23.2-13A (Procedure A), A23.2-23A, A23.2-24A, A23.2-29A	180
Potential expansivity of aggregates	CAN/CSA A23.2-14A	24 months
Detection of alkali-silica reactive aggregate by accelerated expansion of mortar bars	CAN/CSA A23.2-25A	12 months
Petrographic examination of coarse aggregate for concrete	CAN/CSA A23.2-15A	180

  1. Additional aggregate analyses must be carried out at the frequencies specified in Table 4-4.6.4-1 [*Material Test Frequency*] during concrete production to confirm that the aggregates continue to meet Project Requirements.

  2. If the aggregate consists of a blend from more than one source, the “fine aggregate sieve” analysis or the “coarse aggregate sieve” analysis, as applicable, must show the gradation of the blended fine or coarse aggregates.

4-4.6.4.4 Trial Batches

A. Prior to first placement of each different mix design of cast-in-place concrete containing silica fume or hydration stabilizing admixtures, or of Class SCC concrete, and at least once each year trial batch(es) must be carried out as follows to demonstrate that the concrete thus produced has the properties required by the concrete mix design.

1. Trial batch(es) must be prepared at least 35 days prior to placement of concrete at Site.

2. Each trial batch must be a minimum of 3 m^3 or 50% of the mixer's rated capacity (whichever is greater).

3. For concrete mixes containing silica fume:

    a. Slump retention must be evaluated at 15, 30, 50, and 70 minutes after batching.

    b. Slump retention after 50 minutes must be at least 50% of that measured at 15 minutes.

    c. At 70 minutes after batching, samples must be cast to determine compressive strength at 7 and 28 days.

    d. Rapid chloride ion penetration, and hardened air void spacing analysis must be carried out in accordance with the requirements of Section 4-3.5.2 [*Additional Requirements for Concrete Containing Silica Fume*] of this Schedule.

    e. Shrinkage of the trial batch concrete must be measured in accordance with CAN/CSA A23.2-21C.

    f. Trial batch concrete must be placed into a 4.5 m x 4.5 m x 0.15 m thick form on grade to assess the mix's workability and finishability.

4. For concrete containing hydration stabilizing admixtures:

    a. The time of initial and final set must be determined.

    b. Workability including slump and air content must be assessed in accordance with ASTM C403/C403M at 30 minutes after batching, quarter points of the design hydration stabilization period and at the design period.

    c. Hardened air void spacing analysis must be carried out in accordance with the requirements of Section 4-3.5.2 [*Additional Requirements for Concrete Containing Silica Fume*] of this Schedule.

    d. Samples must be cast to determine compressive strength at 3, 7, and 28 days.

    e. Trial batch(es) containing silica fume must also meet the requirements of Section 4-3.5.2 [*Additional Requirements for Concrete Containing Silica Fume*] of this Schedule for rapid chloride ion penetration.

B. For mass concrete mixes:

1. The predicted adiabatic temperature rise of the class of concrete must be reported in the concrete mix design review letter. The method used to determine the predicted adiabatic temperature rise must be in accordance with CSA A23.1 Annex T, Subsection T.3.2.2, Adiabatic Testing, or CSA A23.1 Annex T, Subsection T.3.2.3, Semi-Adiabatic Testing and conducted during the trial batch.

2. Workability retention including, slump and air content, must be assessed at 15 minutes after batching, quarter points of the time of haul and at the end of time of haul. Testing to determine setting characteristics, and compressive strength at 3, 7, and 28 days must be completed. If the mass concrete mix has a hydration stabilizing admixture, trial batching to assess workability retention must be in accordance with Subsection 4.4.4.2, Hydration Stabilized Concrete Mixes.

3. Trial batches for Class SCC concrete:

    a. Fresh concrete must be tested in accordance with the tests identified in Table 4-4.6.4-2 [*Fresh Class SCC Concrete Parameters*].

    b. Hardened air void spacing factor must be determined in accordance with ASTM C457 modified point-count method at 100 times magnification. The average of all tests must not exceed 230 µm with no single test greater than 260 µm. When only two test values are used to calculate the average air void spacing, no test must exceed 230 µm.

4. For Class SCC Concrete mixes:

    a. The requirements of CSA A23.1, Table 22 must be met.

    b. The Maximum Hardened Visual Stability Index (HVSI), determined in accordance with ASTM C1611 must be one.

    c. Hardened air void analysis must be carried out in accordance with the requirements of Section 4-3.5.2 [*Additional Requirements for Concrete Containing Silica Fume*].

    d. A full scale mock-up using a form with characteristics similar to that which will be used on-site must be cast to verify the self-consolidating characteristics and to show that the mix will result in a homogenous structure without segregation.

Table 4-4.6.4-2 Fresh Class SCC Concrete Parameters

Parameter	Minimum Requirement	Maximum Requirement
Slump Flow (CSA A23.2-19C) Flow t-50 cm Time VIS Index	 500 mm Two sec Zero	 800 mm Seven s One
J-ring (CSA A23.2-20C)	-	Difference between slump flow and J-ring slump flow < 25 mm
L-box (MTO LS-440)	h_2/h_1=0.8	h_2/h_1=1.0
Column Separation (ASTM C1610)	-	Static Segregation ≤ 10%

5. The design length of the hydration stabilization period must be the difference between the anticipated haul time and the allowable haul time as specified in Section 4-4.6.7 [*Time of Placing*] of this Schedule or that required by structural or mass concrete pour considerations.

C. For cast-in-place concrete containing silica fume or hydration stabilizing admixtures, only those concrete mix designs for which the trial batch(es) demonstrate compliance with the applicable Project Requirements must be used.

4-4.6.5 Measurement of Materials

A. All constituent materials of cast-in-place concrete must be accurately measured and batched such that the material properties of each concrete batch comply with the properties assumed by the concrete mix design.

1. All constituent materials of cast-in-place concrete must be accurately measured and batched in accordance with the requirements of CAN/CSA A23.1.

2. Air entraining agent and other admixtures must be added to the mix in a water-diluted solution.

3. For mix adjustments at the Site, facilities to control the amount of superplasticizer and air entrainment admixtures must be provided so that the required tolerances are met.

4-4.6.6 Mixing Concrete

A. All materials for the concrete must be charged concurrently at the proportions which satisfy the mix design.

1. Variations in water cement ratio from the applicable concrete mix design is not permitted.

B. All concrete must be mixed thoroughly until it is uniform in appearance, with all constituent materials uniformly distributed.

1. Mobile continuous mixers or other such volumetric concrete supply equipment are not permitted.

2. All joints, valves, and other parts must be maintained so that there is no leakage of water into the mixer drum. Mixers that do not have an accurately working and dependable water gauge must not be used.

3. Air entraining agents and other admixtures must be placed in the mixer after the initial water is in the mixer drum but before any other materials are added. Superplasticizer must be added after initial mixing and in accordance with the superplasticizer manufacturer's recommendation.

4. In no case must the mixing time per batch be less than one minute for mixers of one m^3 capacity or less. The "batch" is considered as the quantity of concrete inside the mixer. The minimum mixing time must be increased by 15 seconds for each additional half m^3 capacity or part thereof. The mixing period must be measured from the time when all materials have entered the mixer drum.

C. Mixers must not be loaded above their rated capacity.

D. Grout and concrete patching materials must be mixed in strict accordance with the manufacturer's recommendations stated on the published product data sheet.

4-4.6.6.1 Truck Mixing

A. Truck mixers must be of the revolving drum type, watertight, and so constructed that the concrete can be mixed at the proportions which satisfy the mix design and ensure uniform distribution of all constituent materials throughout the batch.

1. The maximum size of batch in truck mixers must not exceed the maximum rated capacity of the mixer as stated by the manufacturer and stamped on the mixer.

2. Truck mixing must commence immediately upon introduction of ingredients into the drum and be continued until the concrete is uniform in appearance, with all constituent material uniformly

distributed, but not less than 70 revolutions, with the mixing rate being in accordance with the manufacturer's recommended rate.

3. When adjustment to the mix by adding air entraining agent or superplasticizer admixtures at the Site is made, the mixer must rotate to ensure homogeneity of the concrete, but not less than 70 additional revolutions, before discharge. Discharge chutes must be kept clean and free from hardened concrete and must be wet prior to use.

4-4.6.7 Time of Placing

A. The maximum placing time allowed for all classes of concrete, other than concrete containing silica fume, including delivery to the applicable Site and discharge must not exceed 90 minutes after batching, unless mix design compatible hydration stabilizing admixtures have been employed to extend the setting time.

   1. The maximum placing time allowed must be reduced in accordance with Good Industry Practice under conditions contributing to quick setting of concrete, including hot weather.

B. For cast-in-place concrete containing silica fume the maximum placing time allowed, including delivery to the applicable Site and discharge, must not exceed 70 minutes after batching unless mix design compatible hydration stabilizing admixtures have been employed to extend the setting time.

   1. The maximum placing time allowed must be reduced in accordance with Good Industry Practice under conditions contributing to quick setting of concrete, including hot weather.

C. The maximum placing time must be measured from the time when any of the mix ingredients have entered the mixer drum, regardless of whether or not the drum is revolving.

4-4.6.8 Delivery

A. The rate of delivery of concrete during concreting operations must be such that unplanned cold joints will not develop in the concrete.

B. Deliver and handle the concrete so as to minimize re-handling and facilitate placing of the concrete without damage to the Structure or the concrete.

4-4.6.9 Discharge Temperature

A. Unless otherwise specified, the temperature of concrete containing silica fume must be between 10ºC and 20ºC at discharge.

B. Unless otherwise specified, the temperature of all other classes of concrete must be between 10ºC and 25ºC at discharge.

   1. For concrete elements with a minimum dimension greater than 1.0 m with a thermal control plan submitted to the City for mass concrete pour, the temperature of concrete must be indicated in the mass concrete thermal control plan reviewed and accepted by the Designer.

4-4.6.10 Inspection and Testing

4-4.6.10.1 General

A. Inspection and testing must be carried out as required to confirm that the concrete has the required properties.

B. Sampling of concrete must comply with CAN/CSA A23.2-1C.

1. When a concrete pump is used to place concrete, sampling must be at the end of the discharge hose unless noted otherwise.

2. Sampling may occur at the truck's chute for the following concrete placement:

   a. For concrete placed under water by tremie methods, sampling may occur at the pump's hopper.

3. The properties of concrete tested at the end of the truck's chute must be verified prior to concrete placement by comparing air and slump test results at the truck chute and at the end of the pump hose.

C. Slump tests must comply with CAN/CSA A23.2-5C and must be carried out on all concrete batches.

D. Air content and density tests must comply with CAN/CSA A23.2-4C and A23.2-6C and must be carried out on all concrete batches.

   1. ACI or CCIL/CSA certified testers with related experience must be utilized to test, the air content, density, slump and temperature of each batch of concrete at the Site.

E. If any batch of concrete fails to meet slump or air content specifications, attempts at mitigation must be limited to adjusting the quantities of superplasticizer and air entraining admixtures at Site. Any concrete batch confirmed to be unacceptable by slump, air content or temperature testing must be rejected.

F. Any concrete from a rejected batch already placed in the Structure must be rejected and immediately removed.

G. The test cylinders must be cast as specified in Section 4-4.6.10.2 [*Test Cylinders*] of this Schedule.

   1. Current summaries of concrete testing results including Structure identification, pour location, cylinder identification, slump, air, and individual and average compressive strengths at 7 days and 28 days must be kept by concrete class for each Structure.

H. The proper storage of all site cast concrete cylinders in accordance with Section 4-4.6.10.2 [*Test Cylinders*] of this Schedule, including cylinders cast by the City, is the responsibility of the Design-Builder and adequate cylinder storage space must be provided prior to any concrete pour.

I. Grout cubes must be tested in compression in accordance with CAN/CSA A23.2-1B by experienced ACI or CSA certified testers. A set of compressive strength cubes must be taken to represent each day's production or 0.25 m^3, whichever is more frequent.

4-4.6.10.2 Test Cylinders

A. Making and curing concrete test cylinders must be carried out in accordance with CAN/CSA A23.2-3C, except that the time for cylinders to reach the testing laboratory must be between 20 and 48 hours.

   1. The test cylinders must be cast-in standard CSA approved heavy duty steel or plastic moulds. Plastic moulds must have a wall thickness of at least 6 mm. For concrete containing silica fume, the ends of cylinders must be ground flat prior to testing.

B. Handling and transporting of the cylinders must be in accordance with CAN/CSA A23.2-3C. No extra laboratory curing time must be allowed for cylinders that are delivered late to the laboratory.

C. Temperature-controlled storage boxes for test cylinders must be provided, as specified in Section 8.3.2.1 of CAN/CSA A23.2-3C for a period of at least 24 hours and for protection of the cylinders from adverse weather and mishandling until removed from the Site.

1. A max-min thermometer must be provided for each storage box and for recording site curing temperatures for all test cylinders.

2. Storage in a portable building which will be used during the first 24-hour storage period is not permitted.

D. Test cylinders must be tested in compression in accordance with CAN/CSA A23.2-9C by an independent CSA certified testing laboratory.

E. If the test cylinders exhibit frost etchings or were stored at temperatures below 10ºC or above 25ºC, or were otherwise mishandled, the concrete represented by the test cylinders must be rejected and replaced unless core testing carried out in accordance with Section 4-4.6.10.3 [*Under Strength Concrete*] of this Schedule confirms the in-situ strength of the cylinder.

1. A strength test must consist of the compression tests of four standard test specimens, sampled, made, cured, and tested in accordance with CAN/CSA A23.2-3C as modified herein. One cylinder must be tested at 7 days. The 28-day test result must be the average of the strengths of the remaining three specimens, except that any specimens in a test showing distinct evidence of improper sampling, moulding or testing, must be discarded and the remaining strengths averaged. Additional cylinders may be cast, at the discretion of the Design-Builder. Additional cylinders used to confirm the strength of structural components must be cured in the same manner as the structural components they represent.

F. Test cylinders for strength tests must be taken from representative batches.

1. For concrete containing silica fume as well as for all concrete used in Transportation Structure deck, deck overlay and deck related flatwork, e.g., sidewalks, SUPs, barriers, curbs, medians, a strength test must be taken to represent each approximate 20 m^3 portion of the concrete pour per structure element except that a minimum of one strength test must be taken for every two loads of concrete.

2. For all other concrete a strength test must be taken to represent each Structure element or portion of the element. On larger pours a strength test must be taken to represent each approximate 30 m^3 portion of the concrete pour except that a minimum of one strength test must be taken for every three loads of concrete.

4-4.6.10.3 Under Strength Concrete

A. Concrete with 28-day compressive strength test results less than 100% of the compressive strengths specified in the applicable final Design Data must be removed and replaced unless otherwise accepted by the City, in its discretion.

B. Where permitted by the City, coring to confirm or contest low concrete strength test results must be performed as follows:

1. the cores must be taken and tested within 7 days of the testing of the 28-day cylinders representing the concrete in question;

2. three cores 100 mm in diameter x 200 mm in length must be taken from concrete represented by each non-compliant strength test previously taken. The cores taken must represent the same batch of concrete as the under strength cylinders under consideration; and

3. cores must be sampled and tested by an independent CSA A283 certified testing laboratory and in accordance with the requirements of CAN/CSA A23.214C. CAN/CSA A23.1, Clause 4.4.6.6.2 "Cores drilled from a structure" must not apply. The average strength of the cores as reported by the independent testing laboratory must constitute a "Strength Test".

C. Submit to the City all core results to confirm or contest low concrete strength test.

D. In cases where the concrete strength, as indicated by the cores, is higher than the strength based on the concrete cylinder results, the core results must be used as the basis for acceptance of the concrete. If the core strengths are lower than the strength from the concrete cylinder tests, the strengths indicated by the cylinder tests must govern.

4-4.6.11 Falsework and Formwork

4-4.6.11.1 General

A. Falsework and formwork must be designed, supplied, installed and removed in accordance with CSA-S269.1 and must also meet all requirements for Temporary Supporting Structures in Applicable Law.

B. All formwork must be of sufficient strength and rigidity to ensure that the concrete when the formwork is removed conforms to the design dimensions and contours shown on the applicable final Design Data.

1. The shape, strength, rigidity, water tightness and surface smoothness of reused forms must be maintained at all times. Any warped or bulged formwork must be repaired or replaced before being used.

2. Where the bottom of the form is inaccessible, removable panels must be provided in the bottom form panel to enable cleaning out of extraneous material immediately before placing the concrete.

3. Formwork must be positioned against hardened concrete to avoid form lines and discontinuities at construction joints.

C. All formwork material must be compatible with meeting the surface finish requirements of Section 4-4.6.23 [*Concrete Surface*] of this Schedule.

D. Except for closure pours, all formwork for concrete segmental construction must be steel.

E. All formwork must be removed from the completed Structure.

F. All falsework must be designed and constructed to provide the necessary rigidity to meet the lines and grades shown in the applicable final Design Data, to account for deflections under load and to support the loads without appreciable settlement or deformation.

G. All falsework and formwork shop drawings must be prepared and authenticated by a Professional Engineer.

4-4.6.11.2 Deck Formwork Supported on Girders

A. Deck formwork supported on girders must be fabricated and installed so that the lines and grades shown in the applicable final Design Data are achieved. Adjustments may be made where necessary to compensate for variances in girder dimensions, positioning, alignment and sweep.

1. Prior to commencing deck formwork, all the girders must be profiled, and the deck concrete thickness and girder haunch dimensions required to achieve the specified grade line must be determined. In the event that actual girder camber values vary significantly from the estimated

values indicated on the applicable final Design Data, the grade line must be raised or lowered accordingly.

B. Formwork support brackets must be designed and installed to ensure no damage to girder flanges and webs.

1. Where support brackets bear against girder webs, the contact surface must be protected with timber or neoprene softeners. No drilling of additional holes, or any other modifications including field welding, must be made to the superstructure elements. Effects of concentrated loads on thin webs must be checked, and where necessary, sufficient means must be provided to distribute or carry such concentrated loads to the supporting flanges or stiffeners.

4-4.6.11.3 Forms for Exposed Surfaces

A. Forms for exposed surfaces must meet the requirements of Section 4-4.6.11.1 [*General*] of this Schedule as well as the following requirements.

1. All non-steel forms for exposed surfaces must be as new material, made of coated form ply consisting of Douglas Fir substrate with resin-impregnated paper overlay and factory treated chemically active release agent.

2. Forms proposed for reuse must be free of holes or patched holes in plywood or form liner surfaces, bulging, delamination, damage, or other imperfections that affect the trueness of the formed concrete surfaces.

3. Forms must be full sized sheets, as practicable.

4. The minimum acceptable forming for all exposed concrete where the pour height is 1.5 m or less must have 18 mm plywood, supported at 300 mm maximum centres. Where the pour height is greater than 1.5 m the minimum acceptable forming for all exposed concrete must have 18 mm plywood supported at 200 mm maximum on centres. The support spacing specified is for the use of new material, closer spacing may be required in case of reused material. Strongbacks or walers placed perpendicularly to the supports must be employed to ensure straightness of the form.

5. The top edges of exposed surfaces must have chamfers formed by chamfer strips that establish a true line for screeding.

6. All form material for exposed surfaces must have all joints and seams filled to produce a seam free surface.

B. All forms for exposed surfaces must be mortar-tight, filleted at all sharp corners, and provide for a bevel or draft at all hardware projections to allow the hardware to be subsequently removed to below the concrete surface.

1. Metal bolts or anchorages within the forms must be so constructed as to permit their removal to a depth of at least 20 mm from the concrete surface.

2. Breakback type form ties must have all spacing washers removed and the tie must be broken back a distance of at least 20 mm from the concrete surface. All fittings for metal ties must be of such design that, upon their removal, the cavities which are left will be of the smallest possible size. Torch cutting of steel hangers and ties will not be permitted.

3. When plastic sleeves and removable inner rods are used, the plastic sleeves must be removed for a distance of 100 mm from the face of the concrete except for curbs, barriers and medians where the entire plastic sleeve must be removed.

4. The cavities inside plastic sleeves must be filled with a non-shrink grout to 75 mm from the concrete surface and cured a minimum 24 hours. The remaining 75 mm of the cavity must then be filled with a concrete patching material from the Alberta Transportation Products List in the OH-V category and placed in accordance with the manufacturer's published product data sheet.

5. For fibre reinforced polymer rods, the rods must be removed a distance of 75 mm back from the face of the concrete and filled with an approved concrete patching material from Alberta Transportation Products List in the OH-V category and placed in accordance with the manufacturer's published product data sheet.

C. Form ties on exposed surfaces must be regularly spaced and must not leave holes larger than 40 mm in diameter on the concrete surface.

D. Form hangers or ties for exposed surfaces of decks and sidewalks, including underside surfaces, must be removable threaded type.

E. All forms must provide for a 20 mm x 20 mm chamfer or fillet at exposed concrete edges unless a larger chamfer or fillet is shown on the applicable final Design Data.

4-4.6.11.4 Protection of "Weathering" Steel Bridge Girders

A. Where steel girders are fabricated of "weathering" steel, rust formation on girder surfaces must be of uniform colour.

   1. All joints between deck formwork and steel members (including interior girders, and diaphragms) must be sealed to prevent leakage of cement paste or concrete. Polyurethane sealant or approved equivalent materials must be used to achieve the seal.

B. If marking or staining of the girders occurs, the marks and stains must be removed, and the girder surfaces restored to a uniform colour.

   1. Should foreign material spill onto any weathering steel despite the protection provided, the contaminated areas must be cleaned off, washed, and sandblasted to remove the contamination. Additionally, should the exterior face of an exterior girder become stained or marked, the entire exterior face of the girder line must be lightly sandblasted and "weathered" so that uniformity of girder colour is achieved.

   2. "Weathering" must be achieved by repeatedly fogging the exterior girder faces with clean water and allowing them to dry. Fogging must leave the girder surfaces wet but not "running wet" and must be repeated when the girders are completely dry.

4-4.6.11.5 Protection of Concrete Work from Staining

A. All concrete Project Work must be protected from staining. Any staining of concrete surfaces must be removed.

   1. Stained concrete surfaces that have received a Class 3 Finish must have the entire surface face of the component sandblasted and the Class 3 Finish reapplied; and

   2. Stained concrete surfaces that have received a Class 2 Finish must have the entire surface face of the component refinished.

   3. There must be no trace of staining after the specified concrete finishing is completed.

4-4.6.12 Handling and Placing Concrete

4-4.6.12.1 General

A. Concrete must be placed while fresh and within the time limits set forth in Section 4-4.6.7 [*Time of Placing*] in this Schedule.

1. All the necessary equipment for any pour must be on-site and proven to be in working condition before the pour commences, with backup equipment on-site. The equipment must be well maintained, suitable in kind, and adequate in capacity for the Project Work.

2. In preparation for the placing of concrete, all sawdust, chips and other construction debris and extraneous matter must be removed from the interior of forms. Temporary members must be entirely removed from the forms and not buried in the concrete.

3. Struts, stays, and braces, serving temporarily to hold the forms in correct shape and alignment, pending the placing of concrete at their locations, must be removed when the concrete placing has reached an elevation rendering their service unnecessary.

B. Concrete must not be placed when construction activities adjacent to the Project Work will impart vibration on the concrete that could, in the opinion of the City, cause damage to the concrete.

C. Re-tempering of partially hardened concrete with additional water is not permitted.

D. The method of concrete placement must not have a negative impact on the concrete properties.

1. Concrete must be placed to not cause segregation of the materials or displacement of the reinforcement.

2. When placing operations, involve the free drop of concrete by more than 1 m, it must be deposited by metal pipes or equivalent.

3. Concrete placing operations must be carried out in a manner that minimizes the accumulation of concrete or concrete paste on concrete reinforcement that is not currently being cast into concrete.

4. After initial set of the concrete, the forms must not be jarred, or strain placed on the ends of projecting concrete reinforcement or have additional concrete worked into it.

5. Concrete placing operations must not work off, or transport concrete directly over concrete already placed, when this concrete is less than 48 hours old, no matter what system of runways, supports or protection is used on the surface of the concrete already placed if it is subjected thereby to live or dead loads.

6. When concrete placing is discontinued, all accumulations of concrete or concrete paste deposited on the projecting concrete reinforcement and the form surfaces must be removed before the concrete or concrete paste sets. Care must be exercised not to injure or break the concrete-steel bond at and near the surface of the concrete, while cleaning the concrete reinforcement.

E. Concrete which would be adversely affected by the presence of free-standing water must be protected by preventing its occurrence.

1. All necessary steps must be taken to prevent free water buildup for the first 24 hours.

2. Water used to keep equipment clean during the pour, or to clean equipment at the end of the pour, must be discharged clear of the Structure and any water channel.

F. Grout and concrete patching materials must be placed in strict accordance with the manufacturer's recommendations stated on the published product date sheet.

4-4.6.12.2 Consolidation

A. Concrete, during and immediately after depositing, must be thoroughly consolidated by internal mechanical vibration.

B. The concrete must be thoroughly worked around the reinforcement and embedded fixtures, and into the corners and angles of the forms. Consolidation must be of sufficient duration and intensity to thoroughly compact the concrete but must not cause segregation.

  1. Consolidation must be done by internal mechanical vibration subject to the following provisions.

     a. Vibrators must be capable of transmitting vibrations to the concrete at frequencies of not less than 4500 impulses per minute

     b. Intensity of vibration must be such as to visibly affect a mass of concrete of 25 mm slump over a radius of at least 0.5 m.

     c. Sufficient number of vibrators must be provided to properly consolidate each batch immediately after it is placed in the forms.

     d. Vibrator operators must be suitably instructed in the use of vibrators, and the importance of adequate and thorough vibration of the concrete.

  2. Vibration must be applied at the point of deposit and in the area of freshly deposited concrete. The vibrators must be inserted vertically and withdrawn out of the concrete slowly. Vibration must be of sufficient duration and intensity to thoroughly compact the concrete but must not be continued so as to cause segregation. Application of vibrators must be at points uniformly spaced and not farther apart than the radius over which the vibration is visibly effective.

  3. Vibration must not be applied directly or through the reinforcement of sections or layers of concrete which have hardened to the degree that the concrete ceases to be plastic under vibration. Vibration must not be used to make concrete flow in the forms over distances so great as to cause segregation, and vibrators must not be used to transport concrete in the forms.

  4. Vibration must be supplemented by such spading as is necessary to ensure smooth surfaces and dense concrete along form surfaces and in corners and locations impossible to reach with the vibrators.

C. Once consolidated, concrete must not be disturbed or stepped into. No additional concrete must be added after consolidation.

4-4.6.12.3 Pumping

A. The operation of the pump must produce a continuous flow of concrete without air pockets, contamination or segregation.

B. The equipment must be so arranged that the freshly placed concrete is not damaged by any form of vibration caused by the pump.

4-4.6.13 Placing Pile Concrete

4-4.6.13.1 General

A. The placement of type “Pile” concrete under water will only be permitted in the event that the Design-Builder has demonstrated that all reasonable attempts at obtaining a dry hole have failed.

4-4.6.13.2 Concrete Placed in the Dry

A. Pile Concrete must be placed in accordance with the requirements of Section 4-4.6.12.1 [*General*] and Section 4-4.6.12.3 [*Pumping*] of this Schedule.

   1. Pile Concrete must be placed by means of a hopper equipped with a centre pipe drop tube. The pipe drop tube must be a minimum of 200 mm in diameter and 2 m long. Concrete may be placed free fall, providing the fall is vertically down the centre of the casing or drilled hole and there are no transverse ties or spacers.

B. Concrete in the upper 3 m of the piles must be consolidated in accordance with Section 4-4.6.12.2 [*Consolidation*] of this Schedule.

4-4.6.13.3 Concrete Placed Under Water

A. Placement of Pile Concrete under water must be in accordance with Section 4-4.6.19 [*Depositing Concrete Under Water*] of this Schedule.

B. All Pile Concrete placed under water must be inspected to confirm the structural integrity of the drilled piles by either:

   1. CLS testing in accordance with the requirements of ASTM D6760 and Sections 4-4.6.13.3 [*Concrete Placed Under Water*], 4-4.6.13.4 [*Qualifications*], and 4-4.6.13.5 [*CLS Results*], or

   2. for piles equal to or less than 900 mm diameter, Thermal Integrity Profile (TIP) testing in accordance with the requirements of ASTM D7949 Method B for drilled piles.

C. CSL testing must be completed within 3 to 21 days after concrete placement.

D. A proposed method for carrying out CSL must be submitted to the City before placing any concrete under water. The proposed method must include the following.

   1. All concrete piles cast under water must be equipped with PVC or steel access tubes with regular internal diameter and free from defects, obstructions and joints, to permit inspection by CSL in order to test for voids or other abnormalities in the concrete. The Design-Builder must supply and install one 50 mm inside diameter tube for each 0.3 m of pile diameter with a minimum of four tubes per pile. Tubes must be installed uniformly and equidistantly around the circumference of the pile such that all tubes are parallel for their full length.

   2. Tubes must be installed a minimum of 40 mm away from vertical bars.

   3. Tubes must be water tight, free from corrosion, have clean internal and roughened external faces to ensure a good bond with the concrete. Tubes may be extended with watertight mechanical couplings and all coupling locations must be recorded.

   4. All tubes must be fitted with watertight shoes on the bottom and removable caps on the top. Tubes must extend to within 150 mm of the pile bottoms and must extend a minimum of 600 mm above the pile tops or where they are accessible. Tubes must be capped to prevent debris from entering the access tubes.

5. Tubes must be secured to the interior of the reinforcement stirrups a minimum of every 1.0 m along the length of the pile.

6. Tubes must be installed parallel to each other and equidistant around the circumference of the pile.

7. The tubes must be installed in a manner that allows the CSL probes to pass through the entire length of the tubes without binding.

E. The installation of reinforcement must not damage the CSL tubes.

F. CSL tubes must be filled with water with a minimum temperature of 4°C prior to concrete placement.

G. A 50 mm diameter core hole must be drilled if the testing equipment cannot pass through the entire length of the CSL tube or if tube de-bonding has occurred.

H. CSL measurements must be made at depth intervals of 50 mm from the bottom of the tubes to the top of each pile. Upon completion of testing and verification of the acceptability of the Pile Concrete, the tubes must be filled with a non-shrink grout from the Alberta Transportation Products List with the same strength and durability as the Pile Concrete.

I. The CSL tubes must not be grouted or any further work performed on the CSL tested pile until it has been demonstrated that the pile is acceptable.

4-4.6.13.4 Qualification

A. Submit to the City the name of a proposed testing agency in accordance with the requirements of Schedule 9.

B. The CSL testing report must include test summaries, results, analyses, and an opinion of the Pile Concrete's integrity and suitability for the intended use.

4-4.6.13.5 CSL Results

A. The condition of the concrete piles must be evaluated based on the results of the CSL testing according to the criteria listed in Table 4-4.6.12.5-1 [*Concrete Condition Rating Criteria*].

B. CSL test results with ratings other than "G" will be considered unacceptable and will result in rejection of the pile unless otherwise accepted by the City, in its discretion.

Table 4-4.6.12.5-1: Concrete Condition Rating Criteria

Rating	Velocity Reduction *	CSL Results
Good ("G")	≤ 10%	Good quality concrete
Questionable ("Q")	>10% & <20%	Minor contamination or intrusion: questionable quality concrete
Poor/Defect ("P/D")	≥ 20%	Deficiencies exist, possible water/slurry contamination, soil intrusion and/or poor quality concrete
No Signal ("NS")	No Signal Received	Soil intrusion or other severe defect absorbed the signal

** From highest measured signal velocity in the comparable zone*

4-4.6.14 Placing Deck, Deck Overlay, Slab and Floor Concrete

4-4.6.14.1 General

A. Deck, deck overlay, slab and floor concrete must be placed and screeded so as to not have a negative impact on the concrete properties, to achieve the lines and grades shown in the applicable final Design Data, and to provide drainage of the concrete surface without ponding.

   1. Prior to placing concrete, concrete substrate surfaces must be brought to a saturated surface dry condition with clean water. Substrate surface must be free of standing water.

   2. Prior to placing concrete, a meeting must be held with those responsible for the pour in attendance to review the procedures and accept the conditions of the pour.

   3. Proper lighting must be provided for night pours.

B. Deck, deck overlay, slab and floor concrete must not be placed when the air temperature is below 5ºC, or expected to fall below 5ºC, or when the air temperature is above 25ºC. It must also not be placed in the event of rain or excessive wind and dust, or under any other conditions harmful to the concrete.

   1. Concrete containing silica fume must be placed between the hours of 6:00 pm and 10:00 am of the following day.

   2. Concrete must not be placed when the evaporation rate exceeds 0.5 kg/m^2/hr. The evaporation rate must be determined using Figure D.1, of CAN/CSA A23.1 – Annex D. The rate of evaporation must be recorded as concrete placing operations progress and all necessary adjustments must be made to ensure the evaporation rate does not exceed the specified limit.

C. The temperature of the concrete during discharge must be between 10ºC and 20ºC.

   1. If the temperature of the mix is maintained below the 20ºC maximum by the inclusion of ice to the mix, it must be done in such a way that does not alter the design water cementing materials ratio.

D. All concrete reinforcement projecting from deck surfaces (barriers, curbs, medians, and adjacent deck pour stages) must be covered during deck concrete placement, consolidation, screeding, and testing operations such that it is not contaminated with concrete or concrete paste.

4-4.6.14.2 Placing/Finishing Machines

A. Except for abutment roof slabs and approach slabs placing/finishing machines must be used to finish all 111 Street Road Bridge deck, deck overlay concrete, and any other span where the top surface is the final exposed surface.

   1. Screeding must be by the following concrete placing/finishing machines or equivalents:

      a. Allen Models: 4836 B, 6036 B, 6048 B

      b. Gomaco Model C450 or C750

      c. Terex Bidwell Models: 2450, 3600, 4800

   2. Two work bridges, separate from the placing/finishing machine, of adequate length to completely span the width of the pour must be provided. The work bridges must facilitate the operations of concrete finishing and placing of filter fabric.

3. Work bridges must be supported parallel to the concrete surface, between 250 mm and 600 mm above the concrete surface and must provide an unobstructed working surface that is wide enough to permit diverse uses concurrently, and rigid enough that dynamic deflections are negligible. Work bridges must include specialized work platforms to facilitate concrete finishing in front of curbs, barriers or medians.

B. The finishing machine and guide rails and guide rail supports must be adjusted so that the height of the screed will finish the concrete to the lines and grades shown in the applicable final Design Data.

1. Steel screed guide rails must be installed to suit the profile of the required surface and to ensure a smooth and continuous surface from end-to-end of the deck or deck overlay. Guide rails must extend beyond the ends of the Structure to accommodate finishing of the entire surface with the finishing machine.

2. The finishing machine must be setup to match the skew angle of the Structure, when the skew angle exceeds 15°C. For skewed Structures on vertical curves, this requirement may be altered to suit actual site conditions.

3. To confirm the adjustment of the machine and guiderails, the screed must be dry-run prior to the pour and clearance measurements taken at span tenth points.

4. An independent check must be performed to confirm the design surface profile, deck thickness and rebar cover and the results documented.

5. Resetting of the machine and/or screed rails must be done as necessary, to obtain an acceptable dry-run. Adjustments to the machine or screed rails must not be done after an acceptable dry-run has been completed.

6. Where screed rails are supported on cantilevered formwork that could deflect under the weight of the fresh concrete and the deck finishing machine, a section of the cantilevered formwork on each side of the deck must be pre-loaded to determine deflections that will occur during concrete placement. The formwork, machine and/or screed rails must be adjusted to compensate for the expected formwork deflection.

C. All guide rails and guide rail supports must be located outside of the finished surface of the pour and must be removed with minimal disturbance to the concrete.

D. All deck and deck overlay concrete must be consolidated in accordance with Section 4-4.6.12.2 [*Consolidation*] of this Schedule even when placing/finishing machines are used.

4-4.6.14.3 Screeding Concrete

A. Concrete must be struck off and screeded so as not to have a negative impact on the concrete properties and to achieve the lines and grades shown in the applicable final Design Data.

B. A roll of concrete must be maintained along the entire front of the screed at all times to ensure the filling and consolidation of the surface concrete.

C. Concrete must be placed as close as practicable ahead of the screed, and at no time more than 3 m in front of the trailing end of the screed.

D. The screed must be moved slowly and at a uniform rate.

E. The direction of the pouring must be from the low end to the high end.

F. Screeding must be completed in no more than two passes. If a placing/finishing machine is used, the screed must not be allowed to run except when screeding is in progress. The screeded surface must not be walked on or otherwise damaged.

G. The concrete thickness must be checked by continually probing the concrete behind the screed.

4-4.6.14.4 Bull Floating/Surface Texturing

A. The concrete surface produced behind the screed and after bull floating/surface texturing must be free from open texturing, plucked aggregate and local projections or depressions.

   1. Bull floating, with a magnesium bull float and surface texturing must follow as close as practicable behind the screed. Competent workers who have completed the ACI Concrete Flatwork Finisher Certification Program or are under the direct supervision of a certified journeyman concrete finisher must be employed to carry out bull floating and surface texturing.

B. The screeded surface must be checked for tolerance with a 3 m long expanded polystyrene straight edge immediately after final bull floating and before texturing or application of evaporation reducer.

C. Evaporation reducer or water must not be worked into the concrete at any time.

4-4.6.15 Concreting Shear Keys and Diaphragms

A. Formwork for shear keys and diaphragms must be designed to accommodate variations in girder dimensions, positioning, alignment, camber and sweep. Girder keyways and diaphragms must be brought to a saturated surface dry condition prior to concrete placement. Saturation with water must not be less than 3 hours. Substrate surfaces must be surface dry and free of water immediately prior to concrete placement.

B. Concrete placed in the keyways must be fully consolidated and troweled smooth and level with the top surfaces of the precast girders. Curing must be in accordance with Section 4-4.6.20 [*Curing Concrete*] for the class of concrete specified. For SLW precast concrete girder shear keys, the specified prebagged concrete patching material must be cured in accordance with the manufacturer's recommendations. The use of curing compounds will not be permitted.

4-4.6.16 Concrete Slope Protection

A. A minimum of 1 week prior to commencing concrete slope protection Project Work, the Design-Builder must submit a detailed layout and forming plan to the Designer for review and acceptance. The detailed layout and forming plan must comply with Alberta Transportation Standard Drawing S-1409, Standard Concrete Slope Protection and these specifications.

B. All thickness dimension specified must be measured perpendicular to the top of the concrete slope protection surface.

C. The slopes to be covered with concrete slope protection must be trimmed and dressed by the Design-Builder to within 150 mm of the lines and grades shown on the Drawings. The Design-Builder must supply and place Des 2 Class 25 crushed aggregate material to the thicknesses shown on the Drawing over the trimmed slopes. Crushed aggregate material must conform to the requirements of Section 4-4.1 [*Backfill*].

D. Where slopes have been constructed by others, and excavation exceeding 250 mm or fill exceeding 150 mm is required due to discrepancies in position of the original surface, excavation beyond the 250 mm tolerance limit and/or fill beyond the 150 mm tolerance limit will be considered to be Extra Work.

E. Concrete for slope protection must be Class C.

F. Reinforcing steel must be plain reinforcing steel and in accordance with Section 4-4.10 [*Concrete Reinforcement*]. The Design-Builder's method of securing and maintaining the reinforcing steel in its specified position location must be submitted to the Designer for review and acceptance a minimum of 1 week prior to commencement of the Project Work.

G. The concrete must be placed in either horizontal or vertical courses, with one course being allowed to cure for at least 12 hours before the adjacent course is placed. Formwork must be provided below and above the reinforcing steel to ensure proper slab thickness, correct positioning, and the formation of proper cold joints between courses. Vertical or horizontal joints must be formed or grooved 50 mm to the depth of the reinforcing steel. All joints must be finished with suitable edging and grooving tools and left unfilled. The concrete surfaces must be given a Class 5 finish prior to edging and grooving. Finishing work must be carried out by competent and experienced personnel only.

H. Backfill along the sides of concrete slope protection must be non-granular, conforming to the requirements of Section 4-4.1 [*Backfill*] and must not be placed until the completed concrete slope protection has been reviewed and accepted by the City.

4-4.6.17 Construction Joints

4-4.6.17.1 General

A. Construction joints must be provided between adjacent or successive lifts of concrete.

   1. Construction joints must be installed in accordance with Alberta Transportation Standard Drawing S-1412, Standard Cast-in-Place Concrete Construction Joints.

   2. Before depositing new concrete on or against concrete that has hardened, the forms must be re-tightened.

B. Construction joints must have concrete reinforcement that intersects the adjoining surfaces.

C. The surface of the construction joints must be thoroughly cleaned to remove laitance, damaged concrete and loose and foreign material. The concrete surface must then be blown clean with compressed air and saturated with water, with all free-standing water removed prior to placing concrete against the joint.

   1. Construction joints between cast-in-place concrete girder segments must be roughened to a minimum depth of 10 mm.

D. Construction joints must be made only where indicated on the applicable final Design Data, unless otherwise accepted by the Designer.

E. The face edges of all joints that are exposed to view must be formed true to line and elevation with a pour strip that is removed before proceeding with subsequent concrete placement.

F. Construction joints must be located to allow a minimum of 60 mm concrete cover on reinforcing running parallel to the joint.

G. The placing of concrete must be carried out continuously from construction joint to construction joint.

H. For Building Structures maximum total length of the pour must be 15 m for any structural element.

4-4.6.18 Concreting in Cold Weather

A. During cold weather adequate protection of the concrete must be provided to prevent freezing and to adequately cure the concrete.

1. Cold weather must include any weather when the ambient air temperature is below 5ºC at time of concrete placement or may be expected to fall below 5ºC during the curing period.

B. In addition to the requirements stated below, all concrete must be cured in accordance with Section 4-4.6.20 [*Curing Concrete*] of this Schedule.

C. A procedure for concreting in cold weather must be prepared and submitted to the City prior to any concrete being placed in cold weather. The following provisions must be incorporated into the procedure:

1. Provisions for maintaining the concrete discharge temperature within the limits specified in Section 4-4.6.9 [*Discharge Temperature*] of this Schedule, including:

   a. Heating all aggregates and mixing water to a temperature of at least 20ºC but not more than 65ºC.

2. Aggregates may be heated using either dry heat or steam. The quantity of mixing water must be reduced as necessary to maintain the mix design water cement ratio.

3. Provisions for heating the formwork, concrete reinforcement, previously placed concrete, soil or any other surface the concrete will be placed against. The preheat must be adequate to ensure that no portion of the fresh concrete freezes when placed against adjacent surfaces, or has curing retarded by cold temperatures and must include:

   a. Raising the temperature of all formwork, concrete reinforcement, previously placed concrete, substrate surfaces, soil and any other surfaces the concrete will be placed against to a temperature of between 10ºC and 20ºC before placing any concrete.

4. Provisions for keeping the concrete temperature above 15ºC for a period of 7 days after placing the concrete (except that for concrete containing silica fume, this 7 day period must be increased to 17 days, 14 days wet curing and 3 days of air drying), including:

   a. enclosing the concrete in such a way that the concrete and air temperature within the enclosure are maintained above 10ºC; and

   b. changes and fluctuations to the temperature of the concrete must not change at a rate exceeding 10ºC per day to that of the surrounding air.

5. Provisions for maintaining a wet cure on all concrete surfaces for a minimum of 7 days with Sika UltraCure DOT curing blankets or an approved equivalent.

6. The use of salamanders, coke stoves, oil or gas burners and similar spot heaters that have an open flame or intense heat is prohibited.

7. The enclosure must be constructed large enough to accommodate the workers and equipment necessary to place, finish and cure the concrete. A minimum clearance of 300 mm must be maintained between the enclosure and the concrete.

8. Fully insulated formwork may be used as an alternative to the provision of further heat during the curing period. Formwork must be fully insulated such that the initial heat of the mix and the heat generated during hydration of the cement, maintain the specified curing conditions throughout the curing period.

9. The system of heating and positioning of steam outlets, heaters, and fans must give a uniform distribution of heat. Adequate ventilation must be provided to provide air for combustion and to prevent the accumulation of carbon dioxide within the enclosure. Heaters must be kept well clear of formwork housing.

10. Provisions for withdrawing protection and heat in such a manner so as not to induce thermal shock stresses and cracking in the concrete, including:

    a. gradually reducing the temperature of the concrete at a rate not exceeding 10ºC per day to that of the surrounding air; and

    b. maintaining the temperature differential between the core of the element and the surface of the element below 20ºC. In addition, the temperature differential between the surface of the element and the ambient air must not exceed 15ºC. Ambient air temperature is defined as the air temperature at mid-height and 300 mm from the surface of the element.

11. Provisions for monitoring and recording internal and surface temperatures of the concrete, ambient air temperature and relative humidity inside the enclosure and for making adjustments to protective measures where necessary to meet the requirements of this Section and Section 4-4.6.20 [*Curing Concrete*] of this Schedule.

    a. Monitoring of concrete temperatures, ambient air temperatures and relative humidity must be carried out at least every 4 hours for the first 72 hours after placing the concrete and at least every 8 hours thereafter except that during the withdrawal of heat monitoring must again be carried out at least every 4 hours.

4-4.6.19 Depositing Concrete Under Water

A. Concrete placed under water must be completed in the presence of the Designer.

B. When placing concrete under water, precautions must be taken to prevent segregation, loss or dilution of cementing material, and the introduction of air into the concrete.

   1. Concrete placed under water must be carefully placed in a compact mass, in its final position, by means of a concrete pump line and/or a tremie system. A properly designed and operated tremie may also be used. The concrete must not be disturbed after being deposited. Still water must be maintained at the point of deposit and the forms underwater must be watertight.

   2. A tremie system must consist of a concrete pump line and/or a hopper connected to a rigid tube. If constructed in sections, the rigid tube must have flanged couplings fitted with gaskets.

   3. The use of non-rigid lines will not be permitted.

   4. The discharge end of the concrete pump line and/or tremie system must be temporarily closed to prevent water from entering the line and lowered to the bottom of the form or pile hole. The tremie tube must be kept full to the bottom of the hopper, and water must be kept out of the line at all times.

   5. When a batch is dumped into the hopper, the flow of concrete must be induced by slightly raising the discharge end, always keeping it in the deposited concrete. The flow must be continuous until the work is completed.

   6. Pumping must then proceed with the end of the discharge line being continually buried no less than 1 m below the surface of fresh concrete at all times, to maintain a seal until the form or hole is completely filled with fresh uncontaminated concrete.

   7. Sufficient tremies must be used to place the concrete under water such that it is not necessary to move any of the tremies from one portion of the pour to another.

   8. The surface of the concrete must be kept as horizontal as is practicable at all times.

C. Concrete must not be placed in water which is below 4ºC or flowing at the point of discharge.

D. Concrete must not be placed under water unless permitted by the City, in its discretion.

E. Dewatering is not permitted while concrete placement is in progress. Dewatering may proceed when the concrete seal is sufficiently hard and strong such that dewatering will not damage the concrete.

F. All laitance or other unsatisfactory material must be removed from the exposed surfaces of concrete placed under water using means which will not damage the remaining concrete.

4-4.6.20 Curing Concrete

4-4.6.20.1 General

A. Freshly deposited concrete must be protected from freezing, abnormally high temperatures or temperature differentials, high winds, premature drying, excessive moisture, and moisture loss through the curing period. This includes protection from freezing during the full duration of wet cure and for 12 hours after the removal of wet cure.

B. All concrete surfaces must be wet cured for a minimum of 72 hours at an average ambient temperature of at least 10°C unless otherwise specified.

1. Concrete surface(s) must be covered with two layers of clean, white coloured filter fabric as soon as the surface will not be marred by so doing. The filter fabric must be kept continuously wet during the curing period. Where the formwork is left in place for 72 hours or more, no additional curing will be required for concrete surfaces covered by the formwork.

2. Sika UltraCure DOT curing blankets or an approved equivalent may be used in lieu of using filter fabric.

C. All flat surfaces such as decks, deck surfaces, slabs and slab surfaces, for concrete not containing silica fume must be wet cured for a minimum of 7 days.

D. The temperature of the centre of in-situ concrete must not fall below 10ºC or exceed 70ºC and the temperature difference between the centre and the surface, as well as the temperature differential between top and bottom surfaces, must not exceed 20ºC.

1. To monitor the temperature of mass concrete, including any concrete pour with a minimum dimension greater than 1.0 m, thermocouples must be installed in the pour as follows:

    a. two thermocouples must be placed at each side face (four total) and two thermocouples placed at the centre; for a total of six per set; and

    b. one set of thermocouples must be placed in each pour for each 2 m of pour length where the pour length is the maximum dimension of the pour.

2. The temperatures must be monitored and recorded every 4 hours for the first 72 hours after concrete placement and every 8 hours thereafter for the remainder of the specified cure period and until 24 hours after the maximum temperature has occurred. Whatever means and actions necessary to ensure that the concrete temperature and the temperature differences within the concrete remain within the limits specified must be taken.

E. The requirements of Table 20 of CAN/CSA A23.1 must apply.

F. “Type 1” curing compound complying with the requirements of ASTM C309 (or ASTM C1315) may be applied to concrete surfaces after wet curing unless otherwise specified.

1. Concrete slope protection must receive two coats of a “Type 2” curing compound meeting the requirements of ASTM C309 (or ASTM C1315). The first coat must be applied immediately after

the concrete has been finished, and the second coat within 3 hours after the application of the first coat. Each application must be at the rate specified by the manufacturer.

G. Prepare and submit the procedure for the wet cure of concrete before any concrete is placed. Details must include information with regards to the type and details of equipment and materials being used, and the work methods/techniques employed to carry out the Project Work. The wet cure procedure must be demonstrated to be adequate and suitable prior to scheduling placement of these classes of concrete.

4-4.6.20.2 Curing Requirements for Concrete Containing Silica Fume

A. The requirements of Section 4-4.6.20.1 [*General*] of this Schedule must apply to the curing of concrete containing silica fume.

B. Wet curing must be maintained for a minimum period of 14 days at an average ambient temperature of at least 10ºC.

  1. When concreting in cold weather, wet curing must be maintained for a minimum of 14 days followed by 3 days of air drying at a minimum temperature of 15ºC for both rehabilitation and new construction projects.

  2. Immediately after final bull floating and/or surface texturing and prior to installation of the wet filter fabric cure system, an evaporation reducer having a monomolecular film-forming compound intended for application to fresh concrete for temporary protection against moisture loss, must be applied by a hand sprayer with a misting nozzle at the manufacturer's recommended concentration and application rate. Evaporation reducer or water must not be worked into the concrete at any time during the finishing operation.

  3. Two layers of white coloured filter fabric must be placed on the fresh concrete surface as soon as the surface will not be marred as a result of this placement. The filter fabric must be pre-wet or a fine spray of clean water must be immediately applied to the filter fabric until the filter fabric is saturated. Edges of the filter fabric must overlap a minimum of 150 mm and must be held in place without marring the surface of the concrete. The filter fabric must be maintained in a continuously wet condition throughout the curing period, by means of soaker hoses or other means. The use of polyethylene sheeting above the filter fabric to reduce moisture loss must only be permitted if the sheeting is manufactured with regular perforations to permit the adequate application of curing water from above and reduce the heat generated by greenhouse effects.

C. For those locations where formwork is removed prior to the completion of the specified curing period the resulting exposed concrete surfaces must be wet cured for the remaining days.

D. The temperatures must be monitored and recorded every 4 hours for the first 72 hours after concrete placement and every 8 hours thereafter for the remainder of the specified cure period and until 24 hours after the maximum temperature has occurred.

  1. Two thermocouples, one in the centre and one at the surface of the concrete, must be supplied and installed for every 100 m^2 of deck.

E. Curb and barrier formwork must be removed such that the concrete is not damaged by removal operations, but no later than 72 hours after concrete placement. Wet curing of the concrete surfaces exposed after formwork removal, must commence immediately after formwork is removed.

4-4.6.21 Dimensional Tolerances

A. Except across the crown, deck surfaces of Transportation Structures must be such that when checked with a 3 m long straight edge placed anywhere in any direction on the surface, there must not be any gap greater than 5 mm between the bottom of the straight edge and the surface of the deck concrete, except for the surfaces of the Blackmud Creek LRT Bridge SUP and sidewalk which must not have a gap greater than 3 mm. Parging or surface patching to correct irregularities will not be permitted.

1. Areas that do not meet the required surface accuracy must be clearly marked out and repaired by the following:

   a. any areas higher than 5 mm (3 mm for the Blackmud Creek LRT Bridge SUP) but not higher than 10 mm above the correct surface must be ground down;

   b. any areas lower than 5 mm (3 mm for the Blackmud Creek LRT Bridge SUP) but not lower than 10 mm below the correct surface, must be corrected by grinding down the adjacent high areas; and

   c. when the deviation exceeds 10 mm from the correct surface, the deck or deck overlay must be replaced for a length, width and depth which will allow the formation of a new deck or deck overlay, of the required quality, in no way inferior to the adjacent undisturbed slab.

2. Grinding must be carried out by a machine, of a type and capacity suitable for the total area of grinding involved, until the surface meets the requirements of this Section 4-4.6.21 [*Dimensional Tolerances*].

B. Elements of Building Structures must meet the requirements of Section 7.5 of CAN/CSA A23.1.

C. Formwork misalignment for visible components which can be viewed within a distance of 6 m must be such that when checked with a 1.2 m long straight edge placed anywhere in any direction on the surface, there must not be any gap greater than 3 mm between the bottom of the straight edge and the concrete surface. The gap for formwork misalignment of all other components must not be greater than 5 mm. Concrete elements with formwork misalignments exceeding the allowable tolerances must be removed and recast.

D. Dimensional tolerances for concrete segmental construction must be in accordance Section 4-4.9.3 [*Dimensional Tolerances*] of this Schedule.

E. The maximum angular deviation of a concrete surface from that shown on the applicable final Design Data must not exceed 0.001 radians.

4-4.6.22 Concrete Deficiencies

A. Concrete Deficiencies such as honeycombs, cavities and related defects must be repaired by the Design-Builder to the satisfaction of the City.

1. Concrete Deficiencies are those areas that are greater than 30 mm in depth or 0.05 m^2 in area. Defects less than 30 mm in depth or 0.05 m^2 in area must be repaired in accordance with Section 4-4.6.23.3 [*Class 1A Modified Ordinary Surface Finish*].

2. Concrete Deficiency repair procedures must be developed and authenticated by a Professional Engineer and submitted to the City prior to the commencement of the repair.

3. As a minimum, the repair procedure for concrete Deficiencies must include removing and replacing the defective concrete with the originally specified class of concrete or a non-shrink patching product on the Alberta Transportation Products List.

4. Repair extents must be saw cut 25 mm deep in neat perpendicular lines and concrete removed to a depth of 35 mm below concrete reinforcement.

5. Repair areas must be roughened to remove all loose material and laitance. Exposed concrete reinforcement must be cleaned and repaired to its original condition.

6. Repair areas must be saturated with water for a period of 24 hours prior to repair concrete placement. Repair areas must be free of standing water immediately prior to concrete placement.

7. Curing must be in accordance with the requirements of the class of concrete.

B. Concrete Deficiencies such as cracks with widths equal to or greater than 0.2 mm must be repaired by the Design-Builder to the satisfaction of the City.

1. After the curing period, the dry concrete surface(s) must be inspected, and cracks identified and plotted. The crack widths must be recorded in mm and the crack lengths in m.

2. All repair procedures must be developed and authenticated by a Professional Engineer and submitted to the City prior to the commencement of the repair.

3. Cracks with widths equal or greater than 0.2 mm may be repaired using the following procedure:

a. Clean and dry cracks with oil free compressed air.

b. Seal cracks with an approved gravity flow concrete crack filler from Alberta Transportation Products List in accordance with the manufacturer’s recommendations. The crack filler must maximize the penetration by taking into consideration the ambient temperature, substrate temperature, viscosity and pot life of the material.

c. The crack filler must have a viscosity less than 105 centipoises.

d. Epoxy injection is required for cracks that extend the full depth of the deck slab, barriers, curbs, and other concrete members or extend partial depth of decks that are cast to grade. The epoxy resin must meet the requirements of ASTM C881/C881M Type IV, Grade 1, Class B or C and have a viscosity less than 500 cP.

C. Deficiencies in girders constructed using segmental concrete construction must be assessed in accordance with Section 4-4.7.10.2 [*Precast Concrete Girder and Girder Segment Deficiencies*] of this Schedule.

4-4.6.23 Concrete Surface

4-4.6.23.1 General

A. Prior to concrete surface finishing, all surfaces must conform to the requirements of Section 4-4.6.22 [*Concrete Deficiencies*] of this Schedule.

B. The finished surface of the concrete must conform to the design grades and lines shown on the applicable final Design Data.

C. Wood or magnesium tools must be used for finishing concrete. Finishing aids are not permitted during any concrete finishing operations.

D. For all finishing operations, concrete must not be disturbed (ex. stepped into) once the final screed has been completed.

E. Building Structure concrete surface finishes must be at the discretion of the Designer unless an architectural finish is required in accordance with Schedule 5, Part 5, Section 5-3.12.1 [*Architectural Concrete*].

F. Transportation Structure exposed concrete surfaces to 600 mm below grade or, in the case of river piers, to 600 mm below lowest water level must receive one of the following finishes.

1. Class 1 Ordinary Surface Finish

    a. All exposed formed concrete surfaces for which other finishes are not specified in the Project Requirements.

    b. All unformed concrete surfaces, such as top surfaces of abutment seats, retaining walls and pier caps.

2. Class 1A Modified Ordinary Surface Finish

    a. Trackway slab plinth surfaces located 200 m away from any Stop or Station Platform end.

3. Class 2 Rubbed Surface Finish

    a. All exposed formed concrete surfaces except for the underside of decks between girders.

4. Class 3 Bonded Concrete Surface Finish

    a. None specified.

5. Class 4 Floated Surface Finish

    a. Top surfaces which are to receive waterproofing membranes and wearing surfaces.

6. Class 5 Floated Surface Finish, Broomed Texture

    a. Exposed unformed top surfaces unless other finishes are specified.

    b. Sidewalks

    c. Concrete slope protection

7. Class 6 Floated Surface Finish, Surface Textured

    a. None specified.

4-4.6.23.2 Class 1 Ordinary Surface Finish

A. Unformed Surfaces

1. Immediately following concrete placement and consolidation, the concrete must be screeded to conform to the required surface elevations, and then floated to ensure that the surface is free from open texturing, plucked aggregate, and local projections or depressions.

2. Concrete surfaces must be such that when checked with a 1.2 m long straight edge placed anywhere in any direction on the surface, there must not be any gap greater than 3 mm between the straight edge and the concrete surface.

B. Formed Surfaces

1. The cavities produced by form ties, and all other holes, honeycomb areas, broken corners or edges and other such Deficiencies, must be thoroughly chipped out, cleaned, and filled with a patching product.

a. The patching product must be on the Alberta Transportation Products List, appropriate for the intended application, and placed in accordance with the manufacturer's recommendations.

b. All repairs must be wet cured for a minimum of 72 hours. Curing compounds are not permitted.

2. All fins and irregular projections must be removed immediately after removal of the forms.

4-4.6.23.3 Class 1A Modified Ordinary Surface Finish

A. Surface finish must be as specified in Section 4-4.6.23.2 [*Class 1 Ordinary Surface Finish*] of this Schedule, with the following additional requirements:

1. Prior to finishing, all lines that do not meet tolerance requirements or surface irregularities must be corrected by grinding. Parging or surface patching to correct irregularities is not permitted.

2. Bug holes and surfaces larger than 19 mm in any direction must be repaired.

3. All holes greater than 9 mm on the face of the chamfer or holes greater than 9 mm that could retain water must be repaired.

4-4.6.23.4 Class 2 Rubbed Surface Finish

A. The finish must have a smooth, uniform and closed texture and must be uniform in colour and texture. All cavities in the concrete must be repaired. Any staining must be prevented or removed.

B. Prior to finishing, all lines that do not meet tolerance requirements must be corrected by grinding. Parging or surface patching to correct irregularities is not permitted.

C. All concrete fins and irregular projections must be removed from all surfaces immediately following the removal of the forms.

D. All surfaces must be thoroughly exposed by brush abrasive blasting. Concrete surfaces of piers within a watercourse may be prepared by diamond grinding wheels or other methods reviewed and accepted by the Designer.

1. Abrasive blast cleaning must be completed to a concrete surface profile 3 in accordance with the International Concrete Repair Institute (ICRI) Guideline.

2. Sack rub and patching materials must be cured in accordance with the manufacturer's recommendation. Curing compounds are not permitted.

3. Patching material used must be from the Alberta Transportation Products List and must be wet cured in accordance with the manufacturer's requirements and for a minimum of 24 hours.

4. The cavities produced by form ties, air bubbles and all other holes, honeycomb areas, broken corners or edges and any other such Deficiencies, must be thoroughly exposed. Surface voids greater than 19 mm diameter but less than 0.05 m^2 area and less than or equal to 30 mm deep must be filled with a non-shrink patching product on the Alberta Transportation Products List.

5. The surface voids less than 19 mm in diameter and less than or equal to 30 mm deep must be filled with a prebagged sack rub material. Sack rub materials must be placed over the entire prepared surface in accordance with the manufacturer's recommendations.

6. When checked with a 1.2 m long straight edge placed in any direction on the concrete surface, the gap between the bottom of the straight edge and the concrete surface must not exceed 3 mm unless specified otherwise.

7. When the patching and sack rub materials have adequately cured, a carborundum stone must be used to finish the surface to a smooth, uniform and closed texture. Any voids or cavities opened during the stone rubbing process must be refilled.

8. After the surface preparation has been completed to the satisfaction of the Designer, the Design-Builder must apply sealer as required in Section 4-4.6.24 [*Type 1c Concrete Sealer*].

4-4.6.23.5 Class 4 Floated Surface Finish

A. The concrete surface must be floated and troweled to produce a smooth surface.

   1. The surface must be manually floated with a magnesium bull float.

4-4.6.23.6 Class 5 Floated Surface Finish, Broomed Texture

A. The concrete surface must be finished to produce a smooth surface.

B. After the concrete has set sufficiently, the surface must be given a transversely broomed finish to produce regular corrugations to a maximum depth of 2 mm.

   1. The surface must be manually floated with a magnesium bull float.

   2. A coarse broom must be used to give the transversely broomed finish.

C. A bronze edging tool must be used at all edges and control joints.

D. Sidewalk and median joints must be installed at the same locations as curb joint, barrier joint and control joint locations to a minimum depth of 1/4 of the slab thickness.

E. Where indicated on the applicable final Design Data, sidewalk surfaces must be laid out in blocks using a grooving tool.

F. When checked with a 3 m long straight edge placed in any direction on the concrete surface, the gap between the bottom of the straight edge and the concrete surface must not exceed 3 mm unless specified otherwise.

4-4.6.23.7 Surface Finish under Bearings and Baseplates

A. Concrete surfaces within grout pad recesses must be bush hammered to a depth of 3 mm including all air voids prior to the installation of bearings, bridgerail posts, luminaire, overhead sign structure or other baseplates. Bush hammering must not occur within 25 mm of anchor rods. Anchor rods must be protected from damage during the Project Work.

B. Concrete surfaces on which bearing baseplate pintels or shim stacks will be placed must be ground to a level, smooth, solid and even surface after bush hammering.

C. When checked with a straight edge placed in any direction on concrete surfaces to receive plain elastomeric bearings, the gap between the bottom of the straight edge and the concrete surfaces must not exceed 1 mm.

4-4.6.24 Type 1c Concrete Sealer

A. The sealer must be applied in accordance with the manufacturer's recommendation but the application rate must be increased by 30% from that indicated on the Alberta Transportation Product List.

B. The substrate of the concrete surface must be a minimum of 5°C.

C. The concrete must be cured for at least 28 days prior to the application of the concrete sealer.

D. The concrete surface must be dry, air blasted to remove all dust and debris prior to the application of the concrete sealer.

E. Concrete sealer must be applied using a minimum of two coats.

F. Asphalt concrete pavement surfaces and other elements must be adequately protected from overspray and run-off during the concrete sealer application.

4-4.7 PRECAST CONCRETE

4-4.7.1 General

A. This Section 4-4.7 [*Precast Concrete*] sets out the requirements for all precast concrete forming part of a Structure including minimum requirements for the supply, manufacture, delivery and erection of prestressed and precast concrete. These requirements are in addition to the requirements of Section 4-4.6 [*Cast-In-Place Concrete*] of this Schedule.

1. Precast concrete units include girders, precast concrete segments, MSE walls panels and precast elements for Building Structures.

2. Precast concrete full depth deck panels are not permitted. Deck on girder slabs using precast concrete partial depth deck panels must meet the requirements of Section 4-3.18 [*Decks*] and Section 4-4.21 [*Precast Concrete Partial Depth Deck Panels*] of this Schedule.

3. For Building Structures precast concrete full depth panels/slabs are permitted.

4. Requirements for pre-tensioning are given in this Section 4-4.7 [*Precast* Concrete] of this Schedule. Requirements for post-tensioning are given in Section 4-4.8 [*Post-Tensioning*] of this Schedule.

5. Additional requirements for precast concrete segmental construction are given in Section 4-4.9 [*Concrete Segmental Construction*] of this Schedule.

4-4.7.2 Supply and Fabrication Standards

A. The precast concrete fabricator must be fully certified by the CPCQA Certification Program in the applicable Product Group classification.

B. The manufacture of prestressed and precast concrete must be in accordance with CAN/CSA A23.4 and the PCI Quality Control Manual MNL-116, with the most stringent of the requirements governing.

C. The fabrication of precast concrete units must be done in an environmentally controlled permanent building capable of supplying and manufacturing products in a well-organized and continuous operation. The building temperature must be maintained between 15°C and 30°C and prevent contamination and/or deterioration of materials.

D. Cold weather concreting of precast concrete units will not be permitted.

4-4.7.3 Engineering Data

4-4.7.3.1 Shop Drawings

A. Shop drawings showing all fabrication details of each precast concrete unit must be prepared prior to fabrication and submitted to the City. As a minimum, the following information must be included in a submission:

1. Properties of all materials used;
2. Dimensional information of all precast concrete units;
3. Concrete reinforcement;
4. Prestressing strand;
5. Steel diaphragms;
6. Miscellaneous steel;
7. Blockouts and voids;
8. Stressing system;
9. Anchorage and hold down devices;
10. Void support system; and
11. Screed rail details.

4-4.7.3.2 Stressing Calculations

A. Stressing calculations showing elongations and gauge pressures as well as the strand release sequence data must be provided to the City for each prestressed concrete unit prior to fabrication.

4-4.7.3.3 Stressing Steel and Jack Calibration Certificates

A. A copy of the load/elongation curve for each lot of stressing steel must be available at the precast concrete fabricator's plant. All prestressing strand load/elongation curves must be legible and in English.

B. Where mill test reports originate outside of Canada or the United States of America, they must meet the requirements of Section 4-4.11.3.4 [*Mill Certificates*]. Jack calibration certificates must be provided with stressing calculation design notes an independent check notes.

4-4.7.3.4 Concrete Mix Design

A. Design all concrete mixes to provide concrete that:

   1. is sufficiently workable, for the applicable placement and finishing requirements;
   2. has sufficient durability to meet the Design Life of the Structure; and
   3. has sufficient strength to meet structural strength requirements.

B. Submit to the City concrete mix design together with applicable material test reports to the City before first placement of such concrete.

   1. The mix design must indicate the design strength, proportions of the constituent materials, type and brand of cement, type and source of supplementary cementitious materials, origin of aggregates and brand names of all admixtures.
   2. The mix design must specify the upper slump limit for the superplasticized concrete at which the mix is stable without any segregation. The slump of the concrete used in the production must be 10 mm below the upper limit identified in the mix design.

3. The mix design, including sampling and testing of aggregates, must be authenticated by a Professional Engineer engaged by an independent concrete testing laboratory certified to CAN/CSA A283. The certifying Professional Engineer must also provide a professional opinion confirming that the concrete mix is suitable for the intended use and can be expected to meet all applicable Project Requirements over the Design Life of the Structure.

4. Current test data fully representing the materials to be used in production and showing conformance to the required standards must be submitted with the concrete mix design for the constituent materials.

5. The minimum air content must be in accordance with CSA A23.1 Table 4 based on the maximum aggregate size used and the maximum air void spacing factor must be 0.23 mm.

6. The concrete mix design information must include one microscopic air void analysis performed by an independent CSA A283 certified testing laboratory to determine the spacing factor of the hardened concrete. If adjustments to the mix design are necessary, the air void analysis must be repeated.

7. The test sample used for the microscopic air void analysis must be made from a trial concrete batch, vibrated into a cylinder mould to represent the level of vibration of the production concrete in the forms.

   a. The trial concrete batch should be performed a minimum of 28 days prior to the placement of concrete.

C. Only the Accepted mix design must be used to cast precast concrete.

   1. Changes in cement type, and/or decreasing cement content must be construed as a change in mix design and will not be allowed.

D. Any proposed modifications made to the concrete mix design must be submitted to the City prior to the use of such modifications.

4-4.7.4 Materials

A. All constituent materials for precast concrete must be selected to provide concrete with sufficient durability to meet the Design Life requirements of the Structure and sufficient strength to meet structural strength requirements.

B. Precast concrete must consist of hydraulic cement, silica fume (if required), aggregates, water and admixtures.

4-4.7.4.1 Portland Cement

A. Portland cement must comply with the requirements of CAN/CSA A3001.

4-4.7.4.2 Water

A. Water for mixing concrete, patching products, concrete finishing materials or mortar must comply with CAN/CSA A23.1 and must be free from harmful amounts of alkali, organic materials and other deleterious substances.

   1. Slurry water, treated wash water or water from shallow stagnant or marshy sources must not be used.

4-4.7.4.3 Silica Fume

A. Condensed silica fume must comply with CAN/CSA A3001, for a Type SF supplementary cementing material.

   1. A compatible superplasticizing admixture must be used together with the silica fume.

   2. Silica fume must have a minimum SiO_2 content of 85%, maximum loss on ignition of 10%, and no more than 1% SO_3 content.

4-4.7.4.4 Aggregates

A. Fine and coarse aggregates must be normal weight and comply with CAN/CSA A23.1 and Section 4-4.6.4.3 [*Aggregate Tests*] of this Schedule.

   1. The maximum coarse aggregate size must be 14 mm.

4-4.7.4.5 Admixtures

A. Admixtures must be compatible with all mix constituents.

   1. Acceptable admixtures are air entraining admixtures, superplasticizing admixtures and water reducing admixtures.

B. Air entraining admixtures must comply with ASTM C260/C260M.

C. Water reducing admixtures and superplasticizing admixtures must comply with ASTM C494/494M.

D. All chemical admixtures must be suitable for use in precast concrete, be supplied by the same manufacturer as the air entrainment agent and be compatible with each other.

E. Calcium chloride, accelerators, retarders or set controlling admixtures and air reducing admixtures are not permitted.

F. No admixtures outside of the above requirements must be permitted without the prior written consent of the City, in its discretion.

4-4.7.4.6 Concrete

A. Concrete must consist of hydraulic cement, condensed silica fume, coarse and fine aggregates, water and admixtures. Concrete strength requirements will be specified on the drawings.

B. 10% silica fume by weight of cement (±0.5%) must be used in all precast concrete.

C. Air content must be in accordance with CSA A23.1 Table 4, based on the maximum aggregate size used.

4-4.7.4.7 The maximum air void spacing of hardened concrete must be 230 µm. Voids and Ducts

A. All void and duct material must remain dimensionally stable during the casting and curing of the precast concrete units.

   1. Voids shorter than 400 mm must be eliminated except when noted otherwise on the applicable final Design Data.

B. Concrete longitudinal cold joints that intersect a draped duct must be avoided.

4-4.7.4.8 Galvanizing

A. Factors contributing to galvanization-induced cracking must be minimized. Galvanizing of steel must be by the hot-dip method, after fabrication, in accordance with ASTM A123/A123M Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products and ASTM A153/A153M Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware.

   1. A smooth finish must be provided on all edges and surfaces, and all weld spatter and welding flux residue must be removed from steel components prior to galvanizing.

B. Cleaning and pickling procedure of high strength ASTM A193/A193M Grade B7 anchor rods must be modified as follows:

   1. Brush blast to remove mill scale and oil after threading ends;

   2. Flash pickle up to five minutes; and

   3. Quick dry prior to hot-dip galvanizing (not stored in flux or acid rinse).

C. Galvanizing repairs must provide a coating that has a minimum thickness of 180 μm, adheres to the member and has a finished appearance similar to that of the adjacent galvanizing.

   1. Galvanizing repair must comply with ASTM A780/A780M, Method A3 “Metallizing” unless the area requiring repair does not exceed 100 mm2 in which case the repairs may comply with ASTM A780 Method A1 “Repair Using Zine-Based Alloy”.

   2. Galvanizing repairs must be tested for adhesion.

   3. Repairs may require complete removal of the galvanized coating and re-galvanizing.

D. Galvanized material must be stacked or bundled and stored to prevent wet storage stain in accordance with the AHDGA publication “Wet Storage Stain”. Any evidence of wet storage stain must be removed.

E. Galvanized contact surfaces of bolted connections must be hand wire brushed to a Class C slip coefficient surface condition. Slip coefficients surface conditions must meet the requirements of Table 10.8 of CSA S6.

4-4.7.4.9 Epoxy Bonding Agents

A. Epoxy bonding agents must comply with Section 8.13.7 of the AASHTO LRFD BCS.

4-4.7.5 Fabrication

4-4.7.5.1 Forms

A. Precast concrete units must be fabricated in steel forms which have sufficient strength and rigidity to ensure that the finished precast concrete units conform to the design dimensions. The forms must be mortar tight and set on a rigid foundation.

   1. The forms must be designed such that they can be removed without damaging the precast concrete unit.

   2. For all “I” or “T” beam members, the side forms must be removed horizontally away from the member by a method that prevents any contact of the form with the top flange after release of the form. The top flange must not be subjected to a vertical force at any time.

3. Holes or voids cast into the top flange of "I" or "T" girders to accommodate deck formwork will not be permitted.

B. Match cast precast concrete segments must be separated carefully to avoid damage to the mating surface between the segments. New cast segments must be carefully separated from the bulkhead forms.

1. A bond breaking material must be used on the previously cast segment to facilitate separation of the segments.

C. Precast concrete panels must be cast flat.

4-4.7.5.2 Stressing Strand

A. Stressing strand must be free from corrosion, dirt, grease, rust, oil and other foreign material that may impede the bond between the steel and the concrete.

1. Stressing strand that has sustained physical damage at any time must be rejected.

2. Stressing strand must be protected at all times from manufacture through to encasing in concrete or grouting.

3. Stressing strand with any broken or damaged wire must be removed and replaced. All stressing strands must be checked for wire breaks and damage before placement of concrete. Stressing strand damage includes nicks, gouges, and indentations.

B. Stressing strand splices must not be placed within a precast concrete unit.

C. Each strand must be stressed to a calculated elongation, and a gauge pressure reading must be taken as a check against the calculated force.

1. During stressing, each strand must be first pulled to a predetermined pre-pull gauge pressure to eliminate any slack and a reference mark must be placed at the front of the stressing jack. A second mark must be placed away from the first with a distance corresponding to the calculated elongation on the stressing sheet. Each strand must then be pulled to the second reference mark and the gauge pressure reading taken.

2. This process may be reversed, i.e., each strand must be stressed to a calculated force (determined by a gauge pressure calibration chart) and the elongation must be measured as a check against the calculated force. During stressing, each strand must be first pulled to a predetermined pre-pull gauge pressure to eliminate any slack and a reference mark be placed at the front of the stressing jack. Each strand must then be stressed to the gauge pressure corresponding to the stressing sheet and a second reference mark be placed at this gauge pressure. The elongation must be the distance measured between the two reference marks.

3. At the completion of tensioning, the two control measurements, force and elongation, must meet the verification requirements of Subsection 5.2.2 of the PCI Quality Control Manual MNL-116.

4. Alternatively, the factors contributing to the difference must be identified and corrected before proceeding. Changes in strand temperature and slippage at strand anchorages must be measured between stressing and concrete encasement. Any changes in strand stress due to these effects must be accounted for in the design.

5. All stressing jacks must have been calibrated within 6 months prior to use.

6. Elongation and tension of each strand during the stressing operation must be documented.

7. Stressing strands must not be stressed for more than 36 hours prior to being encased in concrete.

D. Stressing strand ends must be protected as required to prevent corrosion of the strands.

1. The prestressed concrete unit ends must have 15 mm deep strand termination recesses formed around the strands, unless otherwise specified. All strands must be cut flush with the bottom of the recesses.

2. The recesses must be filled flush with the ends of the units with a moisture insensitive epoxy paste adhesive meeting the requirements of ASTM C881/C881M, Type IV, Grade 3, Class B or C. The paste must be grey in colour.

4-4.7.5.3 Void and Duct Placement

A. Voids and ducts must be tied and securely held in their required positions to prevent movement. Continuous ducts must align precisely.

B. The ends of voids must be sealed. Voids found to be distorted, damaged or of insufficient strength must be rejected.

1. Blow holes caused by air expanding within the voids and rising to the surface, must be repaired when the concrete is in the plastic state.

4-4.7.5.4 Lifting Hooks

A. Lifting hooks must not be located on exposed panel surfaces.

4-4.7.5.5 Identification of Precast Concrete Units

A. The fabricator’s name, year of manufacture, unit serial number and design loading must be cast into the bottom of the precast concrete units in 50 mm letters approximately 1.0 m from the precast concrete unit end.

4-4.7.5.6 Concrete Measuring, Mixing and Placing

A. All constituent materials of precast concrete must be accurately measured, mixed and placed such that the material properties of each concrete batch comply with the properties assumed by the concrete mix design.

B. The procedures outlined in ACI Standard 304, Guide for Measuring, Mixing, Transporting and Placing Concrete must be followed.

1. The time from initial mixing of the concrete until placing the concrete in the forms must not exceed one hour.

2. The elapsed time between the successive placements of concrete onto previously placed concrete must not exceed 45 minutes.

4-4.7.5.7 Concrete Temperature

A. The concrete temperature must be between 10ºC and 30ºC at the time of placing concrete in the forms.

4-4.7.5.8 Camber Hubs

A. Three camber hubs must be placed in each precast concrete girder, located along the centreline of the girder at the midpoint and 150 mm from each end.

   1. The girder camber at the midpoint of each girder must be recorded within 24 hours of girder de-stressing.

   2. The camber hubs must consist of 10 mm galvanized bars, of sufficient length to project vertically 10 mm above the riding surface.

   3. The members must be stored in such a manner as to provide access for measuring camber.

4-4.7.6 Inspection and Testing

A. Inspection and testing must be carried out as required to confirm that the concrete has the required properties.

B. Sampling of concrete must comply with CAN/CSA A23.2-1C.

C. Air content and density tests must comply with CAN/CSA A23.2-4C and A23.2-6C.

D. Air void determination testing must comply with CAN/CSA A23.2-17C.

E. The City must be afforded full and safe access for any independent testing and inspection of the precast concrete units at any time. The following equipment must be provided by the Design-Builder at the time of testing or inspection:

   1. cylinder storage box with temperature control and a max/min. thermometer, in accordance with CAN/CSA A23.2-3C; and

   2. a calibrated weigh scales.

4-4.7.6.1 Test Cylinders

A. Test cylinders must be cast and tested to determine the 28-day compressive strength.

   1. Samples for testing must be taken from the fresh concrete being placed in the forms at the rate of one set of cylinders for every 20 m^3 of concrete cast continuously. A set must consist of a minimum of three cylinders.

   2. Making and curing concrete test cylinders must comply with CAN/CSA A23.2-3C.

   3. Testing of concrete cylinders must comply with CAN/CSA A23.2-9C.

   4. Testing must be conducted by an independent CSA certified testing laboratory.

4-4.7.6.2 Strength Tests

A. A “Strength Test” must be the average of the 28-day strengths of the three cylinders (one set).

   1. Continuous casting must mean no break in the casting longer than one hour.

B. Test cylinders for “Release Strength Tests” must be cast and tested to prove that the required release strength as stated on the applicable final Design Data has been attained prior to release of the stressing strand.

1. When one or more units are cast continuously, at least two cylinders must be taken from the concrete of the last unit poured to represent the release strength for all units. These cylinders must be cured with the unit. Only testing of the first cylinder will be necessary if the required release strength is obtained.

4-4.7.6.3 Under Strength Concrete

A. Concrete with 28-day “Strength Test” results less than 100% of the compressive strengths specified in the applicable final Design Data must be removed and replaced unless otherwise consented to by the City, in its discretion.

B. When permitted by the City, coring to confirm or contest low concrete strength test results must be performed as follows:

   1. the cores must be taken and tested within 7 days of the testing of the 28-day cylinders representing the concrete in question;

   2. three 100 mm diameter cores must be taken for each non-compliant strength test previously taken. The cores taken must represent the same batch of concrete as the cylinders under consideration;

   3. cores must be tested by an independent CSA certified testing laboratory and in accordance with the requirements of CAN/CSA A23.2-14C. CAN/CSA A23.1, Clause 4.4.6.6.2 “Cores drilled from a structure” must not apply. The average strength of the cores as reported by the independent testing laboratory must constitute a “Strength Test”;

   4. the core test will represent all precast concrete units represented by the strength test;

   5. alternatively, core tests for strength tests may be taken from each of the other units in question, in which case each of these strength tests will then represent a unit.

C. Submit all core results to confirm or contest low concrete strength test.

D. In cases where the concrete strength, as indicated by the cores, is higher than the strength based on the concrete cylinder results, the core results must be used as the basis for acceptance of the concrete. If the core strengths are lower than the strength from the concrete cylinder tests, the cylinder tests must govern.

4-4.7.7 Release of Stressing Strand

A. Stressing strand must not be released until the specified concrete release strength is attained.

B. Release of the strands must be in accordance with the required de-stressing sequence.

C. Major honeycombs and spalls must be reported to the Designer prior to release of the prestressing strands.

4-4.7.8 Curing

4-4.7.8.1 General

A. All precast concrete units must be cured at an elevated temperature. The curing of precast concrete units must be in accordance with CAN/CSA A23.4 unless otherwise specified.

B. Precast concrete units must be protected from thermal shock at all times until fully cured.

1. The ambient curing temperature must be increased at a rate not exceeding 20ºC per hour until a maximum temperature of not more than 60ºC is attained.

2. After curing, the temperature of the units must be reduced at a rate not exceeding 10ºC per hour until the temperature of the concrete has fallen to within 10ºC of the ambient temperature outside the enclosure.

4-4.7.8.2 Curing Prestressed Concrete

A. Curing in the Form

1. The initial application of heat must commence only after the last of the freshly placed concrete has attained its initial set.

2. Heat must not be applied directly to the concrete but by a method that will produce a consistent ambient temperature throughout the entire form and enclosure.

3. The increase in temperature and the holding temperature must be monitored and permanently recorded on a chart at a minimum of three quarter points along the form.

B. Curing After Removal from the Form

1. Upon removal from the form the unit must be cleaned, patched and finished within a period not exceeding 12 hours.

2. The unit must be placed in a manner that will facilitate any clean up or repair work, and that will allow full inspection of all surfaces.

3. Within 24 hours of removal from the form, the unit must be placed within an enclosure, for curing.

4. The curing enclosure must provide a minimum of 150 mm of free air space between the concrete surfaces and the coverings. Flexible coverings must be secured to prevent any moisture loss.

5. The difference in ambient air temperature adjacent to the concrete at different locations within the enclosure must not exceed 10ºC at any time.

C. The curing process must be continued for a period of at least 4 days with one of the following methods:

1. Steam Curing

   a. The steam must be in a saturated condition maintaining an atmosphere of 95% to 100% relative humidity and a uniform ambient temperature between 40ºC and 60ºC.

   b. Steam jets must not directly impinge on the concrete surfaces.

   c. For days with periods of 4 or more hours within a 24 hour period, where measured temperature or humidity levels do not meet the required limits, these days will not count as a full day of steam cure. An additional full day of steam cure beyond the specified 4 days will be required for each non-compliant day.

2. Curing with Continuous Misting and Heat

   a. The enclosure must be heated to a temperature of between 40ºC and 60ºC at a relative humidity of 95% to 100%.

b. A sufficient number of atomizing misting nozzles must be strategically located to produce a fine mist with 95% to 100% relative humidity in the enclosure.

c. The water must be preheated to a temperature which will produce a misting temperature compatible with the ambient temperature.

d. The enclosure must be heated with radiant heaters.

e. Dry heat must not touch the concrete surface at any time. A control system must be installed to shut off the heat when the humidity level drops below 95% in the enclosure.

f. Should the temperature in the concrete rise above 40ºC without the misting, the unit will be rejected.

3. Two continuously recording thermometers and two continuously recording hygrometers must be provided for each curing enclosure to monitor the concrete and curing rates. All time-temperature and time-humidity recordings must be clearly shown on a graph.

4-4.7.8.3 Curing Non-Prestressed Concrete

A. Curing of all non-prestressed concrete must be in accordance with one of the following methods unless otherwise specified:

1. Elevated Temperature Curing

   a. Upon removal from the forms the units must be cleaned, patched, finished and elevated temperature cured for 4 days in accordance with Section 4-4.7.8.2 [*Curing Prestressed Concrete*] of this Schedule.

2. Moist Curing

   a. Upon removal from the forms the units must be cleaned, patched, finished, and ready for inspection within a period not exceeding 12 hours.

   b. Patching must be performed with a product on the Alberta Transportation Products List and at an ambient temperature of between 15ºC to 25ºC.

   c. After completion of patching and finishing and within 24 hours of removal from the form, the unit must be moist cured at an ambient temperature of not less than 15ºC for a minimum period of 7 days.

   d. Two layers of white coloured filter fabric must be placed on the concrete and kept in a continuously wet condition throughout the curing period by means of soaker hoses or other means unless otherwise specified.

   e. Curing with filter fabric and water must be maintained for a minimum period of 7 days.

B. Curing for MSE wall panels must also conform to the following requirements:

1. Saturation of the face of the panels in preparation for the repair of surface cavities must begin immediately after stripping. During repair of surface cavities, and up to the start of elevated temperature curing or moist curing, panels faces must be kept in a continuously wet condition.

2. As an alternative to moist curing with filter fabric, panels may be moist cured in an enclosure with a controlled temperature and humidity environment such that all exposed concrete surfaces remain saturated for the duration of the curing period. If stacked during curing, sufficient space must be maintained between panels to permit airflow and inspection of surfaces.

4-4.7.9 Dimensional Tolerances

4-4.7.9.1 General

A. Precast concrete unit surfaces must meet the requirements of Section 4-4.7.9 [*Dimensional Tolerances*] of this Schedule.

4-4.7.9.2 Dimensional Tolerances of Precast Concrete Girders

A. The maximum dimensional deviation in mm, of precast concrete girders from the dimensions shown on the applicable final Design Data must not exceed dimensions in Table 4-4.7.9-1 [*Dimensional Tolerances of Precast Concrete Girders*].

Table 4-4.7.9-1 Dimensional Tolerances of Precast Concrete Girders

Length	± 20 mm x length (m) ÷ 50
Width	± 3 mm
Depth	± 5 mm
Camber	± 20 mm x length (m) ÷ 50
Sweep (NU Girders)*	1 mm/m
Sweep (Other Girders)*	deviation from true, 20 mm x length (m) ÷ 50
Projection of Stirrups above Top of Girder	± 12 mm
Bearing Areas	out-of-flatness of bearing areas, 3 mm
Bulkheads	warpage or tilt of ends, 5 mm
Barrier Anchor Bolts	out of line, 5 mm
	in spacing, 5 mm
	in projection, 5 mm
Dowel Holes	out of plumb, 5 mm
Void Location	surface to void dimension, ± 15 mm after casting

** Measured in the plant immediately prior to shipping to Site.*

4-4.7.9.3 Dimensional Tolerances of Precast Concrete Girder Segments

A. Dimensional Tolerances for precast concrete segmental construction must be in accordance with Section 4-4.7.9 [*Dimensional* Tolerances] of this Schedule.

4-4.7.9.4 Dimensional Tolerances of Precast Concrete MSE Wall Panels

A. The maximum dimensional derivations of precast concrete MSE wall panels from the dimensions shown on the applicable final Design Data must meet the requirements of CAN/CSA A23.4.

B. The variation in panel face trueness for any line across a panel face from a straight edge must be no more than 2 mm over 1 m.

4-4.7.10 Concrete Deficiencies

4-4.7.10.1 General

A. Concrete Deficiencies such as cracks, honeycombs, spalls or other defects must be repaired as required to restore the concrete to its initial intended condition as determined by the City acting reasonably.

B. Repairs to all concrete Deficiencies must be carried out in accordance with this Section 4-4.7.10 [*Concrete Deficiencies*].

C. All repair procedures must be developed and authenticated by a Professional Engineer prior to the commencement of the repair.

D. All repairs must be completed prior to curing of the unit and at an ambient temperature of 15ºC to 30ºC. The unit must be protected from dehydrating prior to curing.

   1. Repair of concrete Deficiencies must be done in a sheltered environment and repairs must not be performed in freezing or windy conditions or in direct sunlight.

4-4.7.10.2 Precast Concrete Girder and Girder Segment Deficiencies

A. In this Section 4-4.7.10.2, the “bearing area” of a girder or girder segment used for precast concrete segmental construction is defined as the portion of the girder bottom flange up to the underside of the web, but not including the transition between the bottom flange and the web, directly above the bearing. The bearing area extends from the end of the girder to 75 mm beyond the inside edge of the bearing. The “anchorage area” of a girder is defined as the full height portion of the girder that is within two times the girder depth from the termination of a stressing strand but is not in the bearing area.

B. Cracks

   1. The following cracks must result in rejection of the girder or girder segment unless otherwise consented by the City in its discretion, but without alleviating or otherwise modifying any of the Design-Builder’s responsibility or liability in respect of such cracks, based on an engineering assessment of the effects of the cracks on the ability of the concrete unit to meet the Project Requirements:

      a. cracks in the bearing area of a girder;

      b. cracks in the anchorage area of a girder exceeding 0.5 mm in width for pre-tensioning anchorage areas and 0.2 mm in width for post-tensioning anchorage areas, or longer than 300 mm; and

      c. cracks outside of the girder bearing and anchorage areas exceeding 0.2 mm in width or longer than 300 mm.

   2. Subject to the City’s consent pursuant to Section 4-4.7.10.2.B.1 [*Precast Concrete Girder and Girder Segment Deficiencies*], all repairable cracks 0.2 mm or greater in width must be repaired by epoxy injection in accordance with the manufacturer’s instructions. Coring must be carried out to confirm the penetration of the epoxy into the crack. The epoxy resin must meet the requirements of ASTM C881/C881M Type IV, Grade 1, Class B or C and have a viscosity less than 500 cP.

   3. All repairable cracks less than 0.2 mm in width must receive two coats of a Type 1c sealer unless the crack will be fully encased in a cast-in-place concrete diaphragm or the precast concrete unit requires a Class 3 finish on the surface where the crack is present.

4. The Design-Builder must immediately notify the City and the Designer, if a crack that has a potential to be a shear crack exceeds 0.15 mm in width and longer than 0.25 times the girder depth. Crack length must be measured along the horizontal axis and a crack will be considered to be a shear crack if inclined at an angle between 30° and 60° from horizontal.

C. Honeycombs and Spalls

1. The following conditions of honeycomb or spall must result in a girder or girder segment being excluded from the works unless otherwise consented by the City in its discretion, but without alleviating or otherwise modifying any of the Design-Builder's responsibility or liability in respect of such conditions, based on an engineering assessment of the effects of the honeycombs or spalls on the ability of the concrete unit to meet the Project Requirements:

   a. any honeycombs or spalls in the bearing or anchorage areas of a girder or girder segment; and

   b. major honeycombs or spalls in areas outside the bearing or anchorage areas of a girder or girder segment. Major honeycombs and spalls are honeycombs and spalls that are more than 30 mm deep or more than 0.05 m^2 in area.

2. Subject to the City's consent pursuant to Section 4-4.7.10.2.C.1 [*Precast Concrete Girder and Girder Segment Deficiencies*], all repairs for honeycombs and spalls must be made using a cementitious material.

4-4.7.10.3 Precast Concrete Panel Deficiencies

A. A panel having any one of the following Deficiencies must be rejected:

1. panels with honeycombing, voids, cavities or spalls when the depth exceeds 30 mm or when the area of defect exceeds 150 mm x 150 mm;

2. panels with cracks that are deeper than 25 mm or wider than 0.3 mm;

3. panels with any crack located parallel to or over the strands or concrete reinforcement;

4. exposed MSE wall panel faces with honeycombing, voids, spalls or broken corners;

5. exposed MSE wall panel faces with any surface cavities greater than 10 mm in diameter;

6. exposed MSE wall panel faces with more than three surface cavities per m^2 with cavity diameters from 5 mm up to 10 mm; or

7. exposed MSE wall panel faces with more than 10 surface cavities per m^2 with cavity diameters from 2 mm up to 5 mm.

4-4.7.11 Concrete Finish

4-4.7.11.1 General

A. Prior to concrete surface finishing, all surfaces must conform to the requirements of Section 4-4.7.10 [*Concrete Deficiencies*] of this Schedule.

B. The finished surface of the concrete must conform to the design grades and lines shown on the applicable final Design Data and be free from open texturing, plucked aggregate and local projections or depressions.

C. Building Structure concrete surface finishes must be at the discretion of the Designer unless an architectural finish is required in accordance with Section 4-4.6.23 [*Concrete Surface*] of this Schedule.

D. The determination of the applicable Transportation Structure exposed concrete surface classification must be in accordance with the list provided in Section 4-4.7.11.1 [*General*] of this Schedule. Based on this classification, the finish must meet the requirements set out in this Section 4-4.7.11 [*Concrete Surface*] of this Schedule.

E. Concrete surfaces that will have field concrete cast against them must be sandblast roughened. The blasting must be sufficient to remove all laitance and uniformly expose the aggregate particles.

4-4.7.11.2 Class 1 Form Surface Finish

A. The finished surfaces must be true and uniform.

B. All fins, honeycomb, irregularities, cavities over 10 mm diameter and other similar defects must be thoroughly chipped out and repaired.

C. All repairs must be saturated with water for a period of not less than 30 minutes and pointed and trued with mortar of a colour which will match the adjacent concrete. Mortar used for pointing must be less than 1 hour old.

   1. After repairs, the finish texture must be equivalent to a steel form finish and not a washed or rubbed finish.

D. The repairs must be cured by placing the repaired precast concrete unit in the curing enclosure for a period of 4 days immediately after patching.

E. All surfaces which cannot be repaired must be finished as specified for Class 2.

   1. This finish is essentially that obtained when concrete has been cast and adequately consolidated in a properly oiled steel form.

4-4.7.11.3 Class 2 Rubbed Surface Finish

A. A Class 2 Finish must be the same as a Class 1 Finish except that all holes, cavities and defects must be repaired so that the finished surface presents a smooth, true, dense, uniformly coloured, and non-stained appearance.

B. All residue of form oil must be removed from the surface.

   1. The concrete surfaces must be thoroughly wire brushed to expose any hole or cavity prior to repairs.

4-4.7.11.4 Class 4 Floated Surface Finish

A. The surface must be floated and troweled to provide a closed, uniformly textured surface without brooming.

4-4.7.11.5 Class 5 Floated Surface Finish, Broomed Texture

A. The concrete surface must be floated and troweled to produce a smooth surface.

B. After the concrete has set sufficiently, the surface must be given a transversely broomed finish to produce regular corrugations to a maximum depth of 2 to 3 mm.

1. Brooming must be done when the concrete has set sufficiently to produce clear, crisp brooming marks which do not sag or slump, without tearing the surface or disturbing coarse aggregate particles.

C. After final brooming the surface finish must be free of porous spots, irregularities, depressions, pockets and rough spots.

D. When measured using a 3 m long straight edge placed anywhere in any direction on the surface, there must not be any gap greater than 5 mm between the bottom of the straight edge and the concrete surface.

E. Edging tools must be used on all edges after brooming.

4-4.7.12 Type 1c Concrete Sealer

A. The sealer must be applied in accordance with the manufacturer's recommendations but the application rate must be increased by 30% from that indicated on the Alberta Transportation Product List.

1. The substrate of the concrete surface must be a minimum of 5°C.
2. Before applying the sealer, the concrete must be cured for at least 14 days. Patches must be cured for at least 3 days.
3. The concrete surface must be dry, free of form oil, and air blasted to remove all dust and debris prior to the application of the concrete sealer.
4. Concrete sealer must be applied using a minimum of two coats.

B. Sealer must not be applied in areas of the girders that will have field concrete cast against them.

4-4.7.13 Surface Roughening

A. The roughening of concrete surfaces in shear key, block out, diaphragm and girder end void locations must be achieved by sandblasting or other means acceptable to the City and the Designer. Concrete surfaces must be roughened to a concrete surface profile (CSP) 6 or 7. Concrete surface profiles must be uniform and in accordance with the International Concrete Repair Institute (ICRI) Guideline No. 310.2R-2013, Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, Polymer Overlays, and Concrete Repair. The roughening must be sufficient to remove all laitance and uniformly expose the course aggregate.

4-4.7.14 Handling and Storage

A. Precast concrete units must be handled and stored in a manner that avoids cracking, warping or any other permanent deformations, staining, chipping, or spalling of the member.

1. Precast concrete units must be handled by means of lifting devices at designated locations.
2. Precast concrete units must be stored clear of the ground on blocking where they will not be exposed to splashing.
3. Precast prestressed concrete units must be maintained in an upright position, on stable foundations.
4. Panels with discoloured or stained exposed surfaces are not permitted.
5. Precast concrete panels must be protected from salt spray during shipping.

6. Precast concrete panels must be stored flat.

4-4.7.14.1 Precast Concrete Segments

A. Care must be excised in the handling of precast concrete segments to prevent damage to them. Handling must only be done using the devices shown on the shop/working drawings for this purpose. Lifting devices incorporated into any segment must be adequate to distribute the handling and erection stresses so as not to damage the segment.

B. Precast concrete segments must be stored level in the deck upright position and must be firmly supported on a symmetrical three-point bearing system under the webs at the location's shown on the shop/working drawings. The storage area of the segments must be of suitable stability to prevent differential settlement of the segment supports, resulting in any unstable storage condition during the entire period of storage. Segments must be stored in sequential order so that the uniform appearance of the segments is readily apparent.

C. Prior to shipment, each precast concrete segment must be inspected for damage. The faces of all match cast joints must be thoroughly cleaned of laitance, bond breaking compound and any other foreign material by wire brushing or light sandblasting. During transport, firm support of the segment must be provided, and the segments must be fully secured against shifting. Upon arrival at the erection site, each segment must again be inspected. Incorporation or utilization of any segments with Deficiencies into the Structure is not permitted unless accepted by the City in accordance with Section 4-4.7.10 [*Concrete Deficiencies*] of this Schedule.

4-4.7.15 Erection of Precast Concrete Units

4-4.7.15.1 General

A. The erection of precast concrete units must not damage the units.

   1. No drilling, coring, nailing, installation of any fastening or anchoring systems, or any other modifications must be made to the precast concrete units.

   2. Lifting forces must be vertical. Precast concrete units must not be erected until after any concrete that supports them has been cured a minimum of 3 days and achieved a minimum of 80% of the 28-day specified concrete strength requirements.

4-4.7.15.2 Transportation, Handling and Storing Materials

A. Girders with webs must be transported with girder webs in the vertical position.

B. Precast concrete elements must be protected from dirt, road salts, slush or other contaminants during transportation, handling and storage.

   1. Precast concrete elements that become contaminated must be cleaned prior to erection or installation.

C. Any precast concrete girder damaged during transportation, handling, storing or erection must be immediately reported to the City.

   1. An engineering assessment report and repair procedure prepared and authenticated by a Professional Engineer experienced in the evaluation and inspection of damaged concrete members must be submitted to the City.

   2. An independent inspection and assessment performed on the damaged member by a Precast/Prestressed Concrete Institute (PCI) level two certified inspector.

D. Stored material must be placed on timber blocking and kept clean, free from dirt, grease and any other foreign matter, and store in a properly drained area. Handling and lifting devices must not mark, damage, or distort members. Softeners must be used at all times where chains, flange hooks, restraint systems, or any other tie down devices are or may come in direct contact with the precast concrete unit during transportation, handling, storage, and erection.

E. Precast concrete girders must be shored and stored in the vertical position.

4-4.7.15.3 Temporary Supporting Structures and Berms

A. Temporary supporting structures and berms for the erection of precast concrete units must be designed and constructed and maintained for the forces which may come upon them.

   1. Drawings for temporary supporting structures and berms must be authenticated by a Professional Engineer.

4-4.7.15.4 Erection of Precast Concrete Girders and Girder Segments

A. A detailed erection procedure for the erection of precast concrete girders and girder segments must be prepared in advance of the scheduled start of erection. The erection procedure must include all drawings and documents necessary to describe the following:

   1. TAS;
   2. access to the work, including temporary access berms and/or work bridges;
   3. details of temporary works and supporting structures, including:
      a. location, elevation, and grade of support bearings;
      b. theoretical top of girder elevations at bearing and splice locations as applicable; and
      c. vertical, horizontal, and longitudinal position adjustment mechanisms.
   4. an as constructed survey of substructure elements (hold point), including:
      a. location and elevation of all bearing grout pad recesses including anchor rod voids;
      b. shim height required at each bearing location; and
      c. longitudinal and transverse measurements between centreline of bearings of all substructure elements.
   5. bearing installation procedure in accordance with Section 4-4.14.10.2 [*Bearing Installation Procedure*];
   6. superstructure layout plan, including installation details of reference lines and markings of substructure and bearing components used to determine final bearing and girder positions, and theoretical top of girder elevations at substructure bearing locations;
   7. type and capacity of cranes;
   8. sequence of work, including position of cranes and delivery trucks;
   9. position of cranes relative to substructure elements such as abutment backwalls, with details of load distribution of wheels and outriggers; geotechnical loads and ground capacity requirements;
   10. lifting device details, location of lifting devices, and lifting forces;

11. girder stabilization details, methods of maintaining girder location and alignment, and details of blocking for girder and bearings;

12. diaphragm and bracing installation schedule and sequence (hold point);

13. bolt tightening schedule and sequence (witness point);

14. post-tensioning procedures, including prestressing strand specifications, jack dimensions, pressures, forces and elongations, and grouting;

15. lift hook cutting sequence and repair procedure for lifting holes and pockets; and

16. method and schedule of temporary supporting structures release and removal.

B. The erection procedure must be authenticated by a Professional Engineer, who must assume full responsibility to ensure that the design is being followed.

C. Precast concrete girder segments must not be erected until they are a minimum of 14 days old and have achieved the minimum specified strength shown on the applicable final Design Data.

D. Erection of precast concrete girder segments must only occur when the substrate temperature of the mating segment surfaces is in accordance with the epoxy manufacturer's requirements.

4-4.7.15.5 Girder Adjustments

A. Girder position, bearing location and bearing elevation must be adjusted to achieve as closely as practicable the lines and grades shown on the applicable final Design Data.

B. Minimize differential camber (girder to girder) and the sweep of the girders by jacking, loading of girders, or winching.

1. Once the girder position has been achieved, temporary attachments must be installed to maintain the position.

2. Maximum dimensional deviation in mm of erected precast concrete girders from that as detailed on the final Design Data must not exceed the following:

a. Sweep (NU Girders): 1 mm/m;

b. Sweep (Other Precast Concrete Units): deviation from true, 20 mm x length (m) ÷ 50.

C. All cracks must be inspected, and locations must be mapped if force is required to bring girders into alignment.

4-4.7.15.6 Erection of Precast Concrete MSE Wall Panels

A. The maximum placing deviations in mm, of MSE wall panels from the locations shown on the applicable final Design Data must not exceed the following:

1. the out-of-flatness of wall surfaces measured in any direction must not exceed 25 mm under a 3 m straight edge;

2. the step in face of adjacent panel edges at joints must not exceed 10 mm;

3. the joint taper must not exceed 2.5 mm/m at any location;

4. the overall out-of-vertical or near vertical alignment of the completed wall must not exceed 4 mm/m of wall height from top to bottom of wall; and

5. the maximum variation in average joint width from the specified nominal width must be 10 mm.

B. To facilitate Construction of the cast-in-place concrete coping, nominal-sized, pre-formed holes in the top row of precast panels are permitted providing the holes are located a minimum 100 mm above the underside of the coping.

4-4.7.15.7 Lifting Hooks and Lifting Holes

A. After the precast concrete units are erected and positioned:

1. all lifting holes must be filled with a concrete patching product listed on Alberta Transportation Products List; and

2. all lifting hooks must be cut off 50 mm below the surface and filled with a concrete patching product listed on Alberta Transportation Products List.

4-4.8 POST-TENSIONING

4-4.8.1 General

A. This Section 4-4.8 [*Post-Tensioning*] sets out the requirements for all post-tensioning and grouting of cable ducts for cast-in-place and precast concrete forming part of a Structure. These requirements are in addition to the requirements of Section 4-4.6 [*Cast-In-Place Concrete*] and Section 4-4.7 [*Precast Concrete*] of this Schedule.

4-4.8.2 Standards

A. Post-tensioning and grouting of cable ducts must comply with the applicable requirements of the following standards:

1. CAN/CSA A23.1/23.2 – Concrete Materials and Method of Concrete Construction

2. CAN/CSA A23.4 – Precast Concrete Materials and Construction

3. Acceptance Standards for Post-Tensioning Systems – PTI M50.1

4. Guide Specification for Grouted Post-Tensioning – PTI/ASBI M50.3

5. Specification for Grouting of Post-Tensioned Structures – PTI M55.1

6. AASHTO LRFD Bridge Construction Specifications (AASHTO LRFD BCS)

4-4.8.3 Qualification

A. The site supervisor responsible for the post-tensioning and grouting operations must be at the site whenever post-tensioning operations are being performed.

B. The site supervisor of post-tensioning and grouting operations must be certified to PTI Level Two Bonded PT Field Specialist.

C. The foreman for each installation and stressing crew must be certified to PTI Level Two Bonded PT Field Specialist.

D. The foreman for each grouting crew must be certified to PTI Level Two Bonded PT Field Specialist.

E. At least 25% of the members of each crew must be certified in PTI Level One Bonded – Field Installation.

4-4.8.4 Engineering Data

A. Shop drawings providing a complete description and details of the post-tensioning system to be used must be submitted to the City prior to installation of the ducts. The shop drawings must include:

1. a description of the specific prestressing steel, anchorage devices, duct material and accessory items to be used;

2. properties of each of the components of the post-tensioning system;

3. details covering assembly of each type of post-tensioning tendon including ducts, inlets, outlets, anchorage system, grout caps, protection system materials and application limits;

4. equipment to be used in the post-tensioning sequence;

5. procedure and sequence of operations for post-tensioning and securing tendons;

6. procedure for release and seating of the post-tensioning steel elements; and

7. parameters to be used to calculate the typical tendon force such as expected friction coefficients, anchor set, and prestressing steel relaxation curves.

B. Stressing calculations showing the prestressing jacking sequence, jacking forces and initial elongations of each tendon at each stage of erection for all prestressing must be prepared prior to stressing.

1. The stressing calculations must include calculations to substantiate the prestressing system and procedures to be used including stress-strain curves typical of the prestressing steel to be furnished, friction losses and seating losses.

C. The grout mix design, including the materials and proportions to be used for the grout, must be submitted before first placement of such grout.

1. The grout mix design submission must also include a description of the grouting procedures to be used as required by Section 4.6.1 of PTI M55.1.

4-4.8.5 Materials

4-4.8.5.1 Prestressing Strand

A. Stressing strand must conform to the requirements of Section 4-4.7.5.2 [*Stressing Strand*] of this Schedule.

B. Grouting operations must be completed within 20 calendar days of the installation of the stressing steel, unless otherwise accepted by the City.

C. Corrosion inhibitor must be used when the stressing and grouting operations are not completed within 20 calendar days of the installation of the stressing steel.

1. The corrosion inhibitor, when required, must be water-soluble and must have no deleterious effect on the steel, grout or concrete; or bond strength of steel to concrete.

D. Where unbonded tendons are used for Building Structures, two dynamic tests must be performed on a representative anchorage and coupler specimen. The tendons must, as a minimum, withstand, without failure, 500,000 cycles from 60% to 66% of its minimum specified ultimate strength and also 50 cycles from 40% to 80% of its minimum specified ultimate strength. Each cycle must be taken as the change from the lower stress level to the upper stress level and back to the lower.

E. Unbonded tendons will not be permitted for use in Transportation Structures. External post-tensioning may be permitted at the discretion of the City.

  1. Different specimens may be used for each of the two tests.

  2. Systems utilizing multiple strands may be tested utilizing a test tendon of smaller capacity than the full sized tendon. The test tendon must duplicate the behavior of the full size tendon and must not have less than 10% of the capacity of the full size tendons.

4-4.8.5.2 Anchorages

A. All stressing steel must be secured at the ends by means of permanent anchoring devices. These devices must comply with CAN/CSA S6, Section 8.4.4.1.

4-4.8.5.3 Ducts

A. Ducts must be capable of withstanding concrete pressures without excessive deformation and must prevent the entrance of cement paste into the ducts during the placement of concrete.

  1. Internal ducts must be corrugated plastic pipe. Smooth plastic pipe must be used only for external ducts except that portions of external ducts in deviation blocks must be galvanized rigid steel pipe.

  2. Ducts must be positioned within ± 5 mm of their vertical and transverse positions. Positive methods must be utilized to ensure that the ducts will not be displaced during concrete placement. Internal ducts must be securely fastened at intervals not exceeding 1000 mm.

  3. The ducts must have sufficient rigidity to maintain the required profile between points of supports.

  4. Specific duct material properties must be as follows:

    a. Galvanized Rigid Steel Pipe: Steel pipe duct must be galvanized steel pipe conforming to the requirements of ASTM A53, Type 3, Grade B. The nominal wall thickness of the pipe must not be less than that of Schedule 40. The pipe must be bent to accurately conform to the alignment of the tendon.

    b. Corrugated Plastic Pipe: Corrugated plastic pipe duct must be manufactured from polyethylene material meeting the requirements of ASTM D3350 with a cell classification of 345464A. The duct must contain antioxidant(s) with a minimum OIT according to ASTM D3895 of not less than 20 minutes. The OIT test must be performed on samples from the finished product. The minimum thickness of the duct must meet the requirements of Table 10.8.3.1 of the AASHTO LRFD BCS.

    c. Smooth Plastic Pipe: Smooth plastic pipe duct must be manufactured from polyethylene material meeting the requirements of ASTM D3350 with a cell classification of 344464A. The duct must contain antioxidant(s) with a minimum OIT according to ASTM D3895 of not less than 40 minutes. The OIT test must be performed on samples from the finished product. The minimum thickness of the duct must meet the requirements of Table 10.8.3.1 of the AASHTO LRFD BCS.

B. Mortar tight inlets and outlets must be provided in all ducts and must have a nominal diameter of 20 mm. They must be provided at least at the following locations:

  1. the anchorage areas;

  2. all high points of the duct, when the vertical distance between the highest and lowest point is more than 500 mm; and

3. at low points of the duct.

C. Inlets and outlets must be provided with valves, caps or other devices capable of withstanding the grouting pressure.

1. The inlets and outlets must be securely fastened in place to prevent movement.

D. Ducts must be protected against ultraviolet degradation, crushing, excessive bending, dirt contamination and corrosive elements during transportation, storage and handling.

1. Ducts must be furnished with end caps to seal the duct interior from contamination and shipped in bundles that are capped and covered during shipping and storage.

2. Supplied end caps with the duct must not be removed until the duct is incorporated into the Structure.

3. Duct must be stored in a location that is dry and protected from the sun.

4. Storage must be on a raised platform and the ducts must be completely covered to prevent contamination.

5. If necessary, ducts must be washed before use to remove any contamination.

4-4.8.5.4 Concrete

A. Concrete must be supplied in accordance with Section 4-4.6 [*Cast-In-Place Concrete*] of this Schedule.

1. The 28-day compressive strength must be a minimum of 50 MPa.

2. The maximum nominal size of the coarse aggregate must be in accordance with CSA A23.1 Subsection 4.3.2.2.

4-4.8.5.5 Grout

A. Grout must be Class C as described in Table 10.9.3-1 of the AASHTO LRFD BCS. The properties of the grout must be as described in Table 10.9.3-2 of the AASHTO LRFD BCS.

1. In addition, a test for wet density must be performed in accordance with the "Standard Test Method for Density" in ASTM C138/138M.

2. Materials with a total time from manufacture to usage in excess of 6 months must be retested and certified by the supplier before use or must be replaced.

3. Prebagged grouts must be packaged in plastic lined bags or coated containers, stamped with the date of manufacture, lot number and mixing instructions.

B. The average minimum compressive strength of three cubes at 28 days must be a minimum of 50 MPa in accordance with CSA A23.2-1B.

C. Grout testing must be performed in the field as follows:

1. Strength Test

a. One strength test must be performed for every four longitudinal ducts, except that a minimum of one strength test must be performed for every girder line.

b. The strength test must be carried out by an independent CSA certified laboratory.

2. Bleed Test

   a. At the beginning of each day's grouting operation, a wick induced bleed test must be performed in accordance with ASTM C940 and with modifications noted in Table 10.9.3-2 of the AASHTO LRFD BCS.

   b. The results of the bleed tests must meet the requirements of Table 10.9.3-2 of the AASHTO LRFD BCS.

3. A Schupack pressure bleed test in accordance to ASTM C1741 and PTI M55.1-12 acceptance criteria can be carried out in lieu of the wick induced bleed test.

4. Fluidity Test

   a. For each tendon, a fluidity test must be performed at both the inlet and the outlet in accordance with the standard ASTM C939/C939M flow cone test or the modified ASTM C939/C939M flow cone test.

   b. The results of the fluidity tests must meet the requirements of Table 10.9.3-2 of the AASHTO LRFD BCS.

5. Mud Balance Test

   a. For each tendon, a mud balance test must be performed in accordance with American Petroleum Institute Mud Balance Test API Practice 13B-1 "Standard Procedures for Field Testing Water-Based Drilling Fluids".

4-4.8.6 Equipment

4-4.8.6.1 Stressing

A. Stressing must conform to the requirements of Section 4-4.7.5.2 [*Stressing Strand*] of this Schedule.

B. Hydraulic jacks and pumps with sufficient capacity must be used for tensioning of strands to produce the forces in the strands shown on the applicable final Design Data.

   1. The forces to be measured must be within 25% and 75% of the total graduated capacity of the gauge, unless calibration data clearly establishes consistent accuracy over a wider range.

   2. The measuring devices must be calibrated at least once every 6 months. The jack and the gauge must be calibrated as a unit. A certified calibration chart must be kept with each gauge.

   3. The pressure gauge must have an accurate reading dial at least 150 mm in diameter.

C. The force induced in the prestressing strand must be measured using calibrated jacking gauges, load cells or a calibrated dynamometer.

4-4.8.6.2 Grouting

A. The grout must be mixed using a high speed shear mixer that is capable of continuous mechanical mixing and producing grout that is free of lumps and undispersed cement.

   1. The water supply to the mixer must be measured by an accurate gauge.

B. The grouting equipment must have sufficient capacity to ensure that grouting of the longest duct can be completed within 30 minutes after mixing.

C. The holding tank must be capable of keeping the mixed grout in continuous motion until it is used.

1. The outlet to the pump must have a screen with 3 mm maximum clear opening.

D. A positive displacement type pump must be used which is capable of producing an outlet pressure of at least 1.0 MPa.

   1. A pressure gauge having a full scale reading of no greater than 2 MPa must be placed at some point in the grout line between the pump outlet and the duct inlet.

   2. A spare fully functional pump must be on-site during all grouting operations.

E. Grout hoses and their rated pressure capacity must be compatible with the pump output and the maximum grout pressure. All connections from the grout pump to the duct must be airtight so that air cannot be drawn into the duct.

F. Standby flushing equipment with water supply must be available at the Site prior to commencing grouting.

4-4.8.7 Construction

4-4.8.7.1 Welding

A. Welding of stressing tendons is not permitted.

B. Stressing tendons must not be used as an electrical “ground”.

C. Where the ends of strands are welded together to form a tendon so that the tendon may be pulled through the ducts, the length of the strands used as an electrical “ground” or 1.0 m, whichever is greater, must be cut off from the welded end prior to stressing.

4-4.8.7.2 Tensioning

A. All ducts must be verified as being unobstructed prior to placing post-tensioning steel.

B. All strands in each tendon must be stressed simultaneously with a multi-strand jack.

   1. The force in the tendons must be measured by means of a pressure gauge and must be verified by means of tendon elongation.

   2. All tendons must be tensioned to a preliminary force as necessary to eliminate any slack in the tensioning system before elongation readings are started. This preliminary force must be between 15 and 25 percent of the final jacking force.

C. Stressing tails of post-tensioned tendons must not be cut off until the record of gauge pressures and tendon elongations has been reviewed by a Professional Engineer.

D. A record of the following post-tensioning operations must be kept for each tendon installed:

   1. project name;

   2. subcontractor;

   3. tendon location and size;

   4. date tendon installed;

   5. tendon pack/heat number;

   6. modulus of elasticity (E);

7. date stressed;
8. jack and gauge identifier;
9. required jacking force and gauge pressures;
10. elongation (anticipated and actual);
11. anchor set (anticipated and actual);
12. stressing sequence;
13. witnesses to stressing operation;
14. grout information (brand name);
15. grout test results;
16. time for grouting each tendon;
17. maximum grout pumping pressure at inlet;
18. date grouted; and
19. identification of any grouting problems encountered, and steps taken to resolve them.

4-4.8.7.3 Concreting

A. The anchorage recesses must be concreted after tensioning but before grouting the tendons.

    1. The concrete surfaces of the anchorage recesses must be abrasive blasted.
    2. The recesses must be thoroughly wetted and covered with a thin cement scrub coat immediately before placing fresh concrete.

4-4.8.7.4 Grouting

A. Grouting must not be carried out when there are any conditions that would be detrimental to the grouting operations including when the ambient air or concrete temperature is or is expected to be below 5ºC or above 25ºC during placing or curing of the grout.

B. All ducts and openings must be clean and free of all deleterious matter that would impair bonding of the grout to the ducts and stressing steel.

    1. After installing the ducts and until grouting is complete, all ends of ducts, connections to anchorages, splices, inlets and outlets must remain sealed at all times.
    2. Grout inlets and outlets must be installed with plugs or valves in the closed position.
    3. Low point outlets must be left open.
    4. All ducts must be thoroughly blown out with compressed oil free air. All inlets and outlets must be checked for their capacity to accept injection of grout by blowing compressed oil free air through the system.

C. All ducts and duct connections must be airtight.

1. Before stressing and grouting, install all grout caps, inlets and outlets and test each tendon with oil free compressed air to determine whether duct connections need repair.

2. Pressurise the tendon to 345 kPa (50 psi) and lock-off the outside air source. Record the pressure for one minute. A pressure loss of 170 kPa (25 psi) is acceptable for tendons up to 45 m long, and a pressure loss of 100 kPa (15 psi) is acceptable for tendons longer than 45 m. If the pressure loss exceeds the acceptable limit, repair the leaking connections, and retest.

D. The grout must be mixed so that it is free of lumps and undispersed cement and complies with the properties specified by the grout mix design. Grout must be passed through a screen with 3 mm maximum clear openings before entering the pump.

E. The duct must be completely filled with grout. Grout must be injected continuously through the duct until no visible signs of water or air are ejected from the outlet.

1. All grout vents must be opened prior to commencement of grouting.

2. Grout must be injected from the lowest end of the tendon in an uphill direction. A fully operational grout pump must be on-site for all pumping procedures. A continuous, one-way flow of grout must be maintained at a rate of 5 to 15 lineal metres of duct per minute.

3. The grouting of each tendon must be completed within 30 minutes of mixing of the grout.

4. The pumping pressure at the injection vent must not exceed 1 MPa.

5. Normal pumping pressure must be between 0.1 MPa and 0.4 MPa measured at the inlet.

6. If the actual pressure exceeds the maximum allowed, the injection vent must be closed, and the grout must be injected at the next vent that has been or is ready to be closed as long as one-way flow is maintained. Grout must not be injected into a succeeding vent from which grout has not yet flowed.

F. For each tendon, immediately after uncontaminated uniform grout discharge begins, a fluidity test must be performed from the discharge outlet.

1. The measured grout efflux time must not be faster than the efflux time measured at the inlet or the minimum efflux time established.

2. If the grout efflux time is not acceptable, additional grout must be discharged from the outlet. Grout efflux time must be tested. This cycle must be continued until acceptable grout fluidity is achieved.

G. In addition to the fluidity test, the grout density must be checked using the mud balance method. The density at the outlet must not be less than the grout density at the inlet.

H. To ensure the tendon remains filled with grout, the ejection and injection vents must be closed in sequence, respectively under pressure when the tendon duct is completely filled with grout. Valves and caps must not be removed until the grout has set.

I. 50 mm deep grout tube termination recesses must be formed around the tubes projecting to the surface above the tendon ducts. After grouting, all tubes must be cut flush with the bottom of the recesses, and the recesses grouted flush with the top of the surface.

4-4.9 CONCRETE SEGMENTAL CONSTRUCTION

4-4.9.1 General

A. This Section 4-4.9 [*Concrete Segmental Construction*] sets out additional requirements for portions of Structures constructed using cast-in-place or precast concrete segmental construction. These requirements are in addition to the requirements of Section 4-4.6 [*Cast-In-Place Concrete*], Section 4-4.7 [*Precast Concrete*] and Section 4-4.8 [*Post-Tensioning*] of this Schedule.

4-4.9.2 Submittals

A. Shop drawings, calculations and manuals which include, but are not necessarily limited to, the items listed in this Section 4-4.9 [*Concrete Segmental Construction*] must be submitted to the City prior to any segmental concrete construction being carried out.

B. Any subsequent deviation from concrete segmental construction methods, materials, or details will not be permitted unless the affected submittals are updated and submitted to the City in advance of use.

4-4.9.2.1 Shop/Working Drawings

A. The shop/working drawings must include all details necessary for the successful completion of all precast and cast-in-place segmental concrete construction. They must clearly identify the methods to be used and identify all items to be cast or formed into each concrete pour. They must include but not necessarily be limited to the following:

1. all contractor designed elements must be authenticated by a Professional Engineer;
2. fully and accurately dimensioned views showing the geometry of each segment including projections, recesses, notches, openings and blockouts;
3. complete details of the segment fabrication system, including the forms, form travelers, temporary supports, falsework, temporary foundations, and geometry control. The total weight and centre of gravity of the form travelers including formwork must be indicated;
4. complete details of concrete reinforcement, post-tensioning ducts, post-tensioning hardware, inserts, lifting and hold down devices, and any other items to be embedded in a segment;
5. details of mild steel reinforcing must be clearly shown as to size, spacing and location including any anchorage reinforcing which may be required by the post-tensioning and stay cable anchorage systems;
6. details of post-tensioning ducts must clearly indicate the size, type, horizontal and vertical profiles, duct supports, grout pipes and concrete covers; and
7. details of all inserts or holes in segments including any necessary localized strengthening and the materials and methods to fill and finish such holes must also be included.

4-4.9.2.2 Construction Manual

A. Prior to preparing the casting and camber curves the construction loads, construction stages and schedule corresponding to the construction sequence must be documented in the form of a "Construction Manual". The Construction Manual must include, but is not limited to the following:

1. A detailed step by step description of the construction of the segments, including a description of all intermediate steps relating to any form travelers, construction equipment, falsework, counterweights, support jacking, stressing of temporary post-tensioning bars, jacking of closures

and cantilever tips, closure operations including any partial stressing across the closure during concrete curing, sequence of tendon stressing including stressing loads and elongations, field survey and alignment control.

2. For precast segments, complete details of the handling, storing and transporting of the segments. These details must include, for each type of segment, the method of lifting (location of any inserts, configuration of lifting devices, etc.) and the method of supporting segments during storage and transportation.

3. Complete details covering equipment to be used for casting segments, providing access for post-tensioning, etc., and all loads to be imposed on any portion of the permanent Structure by the construction equipment, temporary supports, and falsework.

4. The Construction Manual must make appropriate reference to the Geometry Control Plan and Procedure. It must include the sequence in which segments and individual components of each segment will be cast.

B. A new Construction Manual must be prepared at any time that there is a deviation from the sequence and schedule of construction contained in the current Construction Manual.

4-4.9.2.3 Design Calculations for Construction Procedures

A. Calculations authenticated by a Professional Engineer must be submitted to the City that show that the loads imposed on the permanent Structure by the temporary construction loads and construction sequence will not adversely affect the permanent Structure, nor exceed allowable stresses during the Construction process.

4-4.9.2.4 Casting and Camber Curves

A. Horizontal and vertical deflection and camber data for each stage of Construction as required to construct the Structure to its final alignment, grade and superelevation must be prepared.

B. Bearing offsets and Structure geometry must be adjusted for time dependent displacements. Data used must account for the effect of the time dependent prestress losses, creep and shrinkage which will occur during the Construction phase and must be consistent with the intended usage described in the Geometry Control Plan and Procedure. The data for the entire Structure, based on the construction sequence, method and schedule, must be prepared prior to commencing concrete segmental construction of the applicable Structure.

C. Construction stage camber data must be prepared in accordance with the casting, post-tensioning and stay cable installation sequence, schedule, construction techniques, loads, introduction or removal of temporary supports, falsework, construction equipment, closure devices, and material properties documented in the Construction Manual.

1. The camber curves must have sufficient accuracy to allow for the determination of control point settings for accurately casting the segments with respect to both horizontal and vertical geometry.

2. The preparation of the camber curves must recognize all deviations and deformations from the final required profile and alignment due to Structure self-weight, future superimposed dead loads, construction loads, post-tensioning and stay cable effects including secondary moments, creep and shrinkage effects, the effects of temperature variations and non-linear pier behavior.

3. Each camber curve must be accompanied by all information (loads, casting and construction sequence, material properties, traveler deflection, etc.) considered in its development.

D. Camber and erection elevation tables must include theoretical elevations and alignment of the geometry control points and form travelers established during casting of each segment and computed at each stage of construction. A summary of elevations for each joint which gives the elevation history of that joint during the various stages of construction must be furnished. Stages for which theoretical positions of control points are to be computed must include:

1. unloaded formwork in position ready to receive concrete;

2. after each concrete segment is placed;

3. after each stage of applying post-tensioning or stay cable forces; and

4. after any change in support conditions.

E. If the construction sequence is changed, camber curves must be prepared in the same manner as required for the original camber curve. The revised camber curve must include the methods(s) and location(s) for transitioning between the current curve(s) in use and the updated curve(s).

F. The camber of the structure must be monitored at each stage according to the Geometry Control Plan and Procedure described below. Corrections must be performed as required to assure proper construction of the Structure to its final alignment and grade.

4-4.9.2.5 Geometry Control Plan and Procedure

A. A Geometry Control Plan and Procedure which indicates in detail how the survey is to be performed and proper casting and construction of the Structure carried out to achieve the lines and grades shown on the applicable final Design Data must be prepared.

B. The Geometry Control Plan and Procedure must provide for regular monitoring of Structure deflections beginning with the addition of the first segments and concluding with the last closure. The Geometry Control Plan and Procedures must include the adjusting procedure to be utilized for each segment, and must also include special adjustment procedures should the segments, as constructed, begin to deviate from the predicted alignment by more than 25 mm.

C. The Geometry Control Plan and Procedure must include the following information:

1. a detailed narrative of the geometry control theory;

2. a detailed narrative of the step by step geometry control procedure;

3. detailed calculation forms; and

4. a set of sample calculations.

D. The Geometry Control Plan and Procedure must address all measuring equipment, procedures, the locations of the control points to be established on each segment and the qualifications of personnel who will carry out geometry control.

1. Personnel who directly supervise layout and geometry control measurements must have previous experience in geometry control techniques for concrete segmental bridges.

E. The Geometry Control Plan and Procedure must cover all geometry control operations necessary for casting and placing the segments and must supplement the Construction Manual.

1. Casting must not commence until after the Geometry Control Plan and Procedure is finalized.

F. A table of elevations and alignments required at each stage of Construction and at all control points must be prepared. Any deviation from the table of elevations and alignment must be corrected so as to prevent the accumulation of deviations.

1. A record of all checks, adjustments and corrections made during Construction must be maintained.

G. During segment casting or placing operations, computer generated graphical plots of the vertical and horizontal "as cast" alignments along each vertical and horizontal control line must be produced and maintained on a daily basis. These plots must use an exaggerated scale in order to clearly highlight variations. These plots must be depicted against both the theoretical geometric vertical and horizontal alignment casting curves on a continuous layout along the entire length of the Structure between expansion joints.

1. A printed copy of this plot must be maintained in good condition at the applicable Site, for use and reference during erection.

H. Immediately after casting or placing of a segment is completed, references for horizontal and vertical control must be established at the leading free end of the segment.

I. Elevations and alignment of segments must be measured at each stage of construction with instruments capable of providing the degree of accuracy necessary to assure that construction tolerances will be met.

1. The alignment and elevations of the segments must be checked from established control at a time that will minimize the influence of temperature.

2. Precaution must be used to guard against possible false readings and corresponding adjustments due to temperature differentials.

3. A minimum of two remote permanent horizontal survey control triangulation points and vertical control benchmarks must be established at each applicable Site. Permanent benchmarks must be established at locations where they will not be disturbed by construction activities. The horizontal control points and benchmarks must be located to be continuously visible from the survey instrument's location.

4. Prior to casting or placing a new segment, the position of the previous segment must be independently verified by two surveys.

J. The segments must be positioned to achieve the final longitudinal alignment, grade and cross slope.

1. Casting of the segments adjacent to the pier table must not begin until the form travelers are properly tied down to the piers by the means provided.

K. If segment positions are not as required, corrections to the geometry must be made to the next segment cast by utilizing the established control points.

1. If measured elevations deviate from the approved table of elevations further casting or placing of segments must be suspended until the cause of the deviation is discovered and a Corrective Action Plan prepared.

4-4.9.3 Dimensional Tolerances

A. The maximum dimensional deviations in mm of cast-in-place and precast concrete segments used in segmental concrete construction from the dimensions shown on the applicable final Design Data must not exceed the values shown in Table 4-4.9.3 [*Dimensional Tolerances of Concrete Segments*].

Table 4-4.9.3 Dimensional Tolerances of Concrete Segments

Width of Web	± 6 mm
Depth of Bottom Slab	± 5 mm
Depth of Top Slab	± 5 mm
Overall Depth of Segment	± 5 mm
Overall Width of Segment	± 6 mm
Length of Segment	± 10 mm but less than 50 mm total per cantilever
Diaphragm Dimension	± 10 mm
Grade of form edge and soffit	± 3 mm per 3.0 m
Ends (deviation from a plane per 6 m width or depth)	± 6 mm per 6 m not to exceed 12 mm
Flat surface (deviation from a plane at any location)	± 1.5 mm per 1.0 m not to exceed a total of 6 mm

4-4.9.3.1 Erection Tolerances

A. The following dimensional tolerances must apply to the erection of cast-in-place or precast concrete segments:

1. the maximum differential between the outside faces of adjacent segments in the constructed position must not exceed 5 mm;

2. transversely, the angular deviation from the theoretical slope difference between two successive segment joints must not exceed 0.001 radians;

3. longitudinally, the angular deviation from the theoretical slope change between two successive segments must not exceed 0.003 radians; and

4. the difference in top of segment elevations at the connection of two adjacent segments (measured perpendicular to the segment surface) and across closure joints must be no greater than 3 mm.

B. Dimensions from segment to segment must be adjusted so as to compensate for any deviations within a single segment so that the overall dimensions of each completed span and the entire Structure will conform to the dimension shown on the applicable final Design Data.

1. The accumulated maximum error must not exceed 1/1000 of the span length or 100 mm whichever is less, for either the vertical profile and/or horizontal alignment.

4-4.9.4 Closure Pours

A. For cantilever segmental concrete construction, the cantilevers must be fixed prior to the closure pour between the cantilevers to prevent rotation or movement of one cantilever relative to the other.

1. The system for locking the cantilevers and forming the closure pour and the procedure for placing the concrete for the closure must be such that the concrete after initial set must not be subjected to tension which could cause cracking.

4-4.10 CONCRETE REINFORCEMENT

4-4.10.1 General

A. This Section 4-4.10 [*Concrete Reinforcement*] sets out the requirements for all concrete reinforcement forming part of a Structure, including minimum requirements for quality, supply, fabrication, handling and placing of plain reinforcing steel, CRR, and stainless reinforcing steel placed in cast-in-place concrete and precast concrete units.

4-4.10.2 Engineering Data

A. Shop drawings showing concrete reinforcement details must be prepared prior to fabrication of concrete reinforcement.

B. Mill test reports must be prepared for each lot of concrete reinforcement delivered to site prior to the placement of any concrete reinforcement.

C. Mill test reports must be provided in English and at a minimum include:

1. Heat number;
2. Date;
3. Location of product;
4. Compliance with production standards;
5. Chemical analysis;
6. Mechanical properties;
7. Pickling process details for stainless reinforcing steel; and
8. Authentication by the manufacturer.

D. Mill test reports originating from a mill outside of Canada or the United States of America must meet the requirements of Section 4-4.11.3.4 [*Mill Certificates*] of this Schedule.

E. The following additional information, as applicable, must be supplied for each lot of stainless reinforcing steel delivered to site:

1. Austenitic grades: Test results verifying compliance with ASTM A262, Practice E; and
2. Duplex grades: Test results verifying compliance with ASTM A1084, Method C by demonstrating no presence of detrimental phases.

4-4.10.3 Fabrication

A. Concrete reinforcement bars must conform accurately to the dimensions shown on the applicable final Design Data and be within the fabricating tolerances detailed in the RSIC Manual of Standard Practice.

B. All hooks and bends must be fabricated using the pin diameters and dimensions recommended in the RSIC Manual of Standard Practice.

C. Fabrication of stainless reinforcing steel must be carried out such that bar surfaces are not contaminated with deposits of iron or other non-stainless steels; or suffer damage due to any cause, including straightening or bending.

D. Stainless steel fabrication facilities must be exclusive to the fabrication of stainless reinforcing steel or in a facility that provides a permanent fixed physical barrier which fully isolates the stainless steel fabrication process.

  1. Fabrication must occur only on equipment dedicated solely to fabrication of stainless reinforcing steel.
  2. All machinery points that come into contact with stainless reinforcing steel bars must consist of hardened steel to a minimum of 35 Rockwell, stainless steel, or nylon.
  3. All racking must be protected with hardened steel to a minimum of 35 Rockwell, stainless steel, nylon or wood.

E. All concrete reinforcement requiring bends must be cold bent at the fabrication facility, unless otherwise approved by the Designer and accepted by the City.

  1. Heating of concrete reinforcement to facilitate bending is not permitted.

F. Concrete reinforcement must be cut by shearing or with fluid-cooled saws.

  1. Torch cutting is not permitted.

G. Concrete reinforcement must be fabricated without laminations or burrs.

H. Stainless reinforcing steel must be pickled to remove all mill scale and surface oxidation.

I. Stainless reinforcing steel must be shot blasted and pickled at the production mill to remove all mill scale and surface oxidation.

4-4.10.4 Shipping, Handling and Storage

A. All necessary precautions must be taken to prevent damage to the concrete reinforcement during shipping, handling and storage.

  1. Concrete reinforcement of differing material types must be stored separately.
  2. All concrete reinforcement must be stored on platforms, skids, or other suitable means of support able to keep the material above the ground surface while protecting it from mechanical damage and deterioration.
  3. On-site storage of concrete reinforcement must not exceed 120 days unless protected with polyethylene sheeting or equivalent protective material.
  4. Concrete reinforcement must be covered and protected at all times during transportation.
  5. Bundles must be handled with spreaders and non-metallic slings.
  6. Lifting of stainless steel reinforcing must be completed with nylon strapping dedicated to stainless reinforcing steel.
  7. Fork trucks used in the handling of coil or straight stainless reinforcing steel must have their forks covered with hardened steel to a minimum of 35 Rockwell, stainless steel, or nylon.
  8. Stainless reinforcing steel bundles must be tied with plastic strapping or stainless steel tie-wire and not with carbon steel or epoxy coated carbon steel strapping.
  9. Polyethylene wrap must be used to fully cover all stainless reinforcing steel bars and bundles for shipping.

10. Stainless reinforcing steel must be covered with a tarp at all times during shipping with tarps dedicated for stainless reinforcing bars.

B. Concrete reinforcement tags identifying the material type must be clearly visible and must be maintained in-place until installation of the material.

4-4.10.5 Placing and Fastening

A. Concrete reinforcement incorporated into the Project Work must be free from loose rust, scale, dirt, paint, oil, concrete, concrete paste, or other foreign materials.

B. Concrete reinforcement must be accurately placed in the positions shown on the applicable final Design Data and must be securely tied and chaired before placing the concrete.

1. Bars must be tied at all intersections except that when the bar spacing is less than 250 mm in each direction, alternate intersections may be tied at these locations.

C. Unless otherwise specified, tie-wire must be manufactured from the same material type as the reinforcing bar being tied.

1. Plastic coated tie-wire may be used where low carbon/chromium reinforcing steel is being placed.

2. Where stainless reinforcing steel is being placed, tie-wire must be stainless steel of any grade listed in Section 4-3.5.3 [*Concrete Reinforcement*] of this Schedule.

D. Concrete reinforcement cover must not be less than that specified on the applicable final Design Data.

1. Supports used to prevent bars from contact with forms or for separation between layers of bars must be of adequate strength, shape and dimension.

E. Bundled concrete reinforcement of two or more bars is not permitted. Splices of bars in bundles must be staggered.

F. Specified distances of concrete reinforcement from forms must be maintained by supports, spacers, or other means.

1. Concrete reinforcement supports must be either plastic or precast concrete. Brick or mortar supports are not permitted.

2. Supports must be staggered and configured to facilitate full concrete consolidation.

3. Precast concrete supports must have the compressive strength, rapid chloride permeability, and air content meeting the specification requirements for the class of concrete being placed. The precast concrete supports must be placed in a configuration that minimizes the geometric size of the precast concrete support and does not adversely affect concrete placement and consolidation processes.

4. Plastic bolster slab supports must be Aztec Strong Back Slab I Beam Bolster- PSBB manufactured by Dayton Superior or equivalent.

5. Bolster slab supports of length not exceeding 100 mm must be used for exposed faces of curbs, medians and barriers.

6. Precast concrete supports must be Total Bond Concrete Supports manufactured by Con Sys Inc or equivalent.

G. Welding of concrete reinforcement is not permitted, except for concrete reinforcement connected to embedded steel plates, provided weldable reinforcing steel is used and magnetic particle testing of 100% of the welds of every embedded plate is carried out in accordance with Section 4-4.11.9.1 [*General*] of this Schedule and all discovered deficiencies are repaired.

H. Field bending of concrete reinforcement, is not permitted, unless otherwise approved by the Designer and accepted by the City.

I. Concrete reinforcement showing signs of damage must be replaced.

4-4.10.6 Concrete Reinforcement Tolerances for Cast-In-Place Concrete

A. Concrete reinforcement for cast-in-place concrete must be placed in conformance with the following tolerances:

1. location, where the smallest dimension of the element is:
    a. 200 mm or less: ±8 mm
    b. larger than 200 mm but less than 600 mm: ±10 mm
    c. 600 mm or larger: ±20 mm
2. lateral spacing: ±30 mm;
3. longitudinal location of bends and ends of bars: ±50 mm;
4. longitudinal location of bends and ends of bars at discontinuous ends: ±20 mm; and
5. the clear distance between reinforcement must not be less than 1.5 times the nominal diameter of the reinforcement, 1.5 times the maximum coarse aggregate size, or 40 mm.
    a. Pile reinforcement clear distance must not be less than five times the maximum coarse aggregate size, or 125 mm.

4-4.10.7 Splicing

A. Concrete reinforcement splices must be staggered unless otherwise specified on the applicable final Design Data.

B. For lapped splices, bars must be placed in contact and tied together while maintaining the minimum required clear distance to other bars and the required minimum distance to the surface of the concrete.

4-4.10.8 Repair of Stainless Reinforcing Steel

A. Individual stainless steel reinforcing bars exhibiting any Deficiencies including any of the following Deficiencies must be repaired or replaced:

1. any location of contamination from grinding or cutting slag;
2. any location of iron contamination greater than 100 mm in length;
3. more than 10 discrete points of iron contamination on bar deformations within any 1 m of bar length;
4. more than 20 discrete points of iron contamination on bar deformations per bar; or

5. more than five discrete points of iron contamination that are not located on bar deformations per bar.

B. Discrete points of contamination are defined as areas of contamination less than or equal to 5 mm^2. If any area of contamination is larger than 5 mm^2, the area must be divided by five to determine the number of discrete points.

C. Bars exhibiting excessive staining must have the contaminants identified by EDXA.

D. Methods for the repair of stainless reinforcing steel bars must be prepared and authenticated by a Professional Engineer prior to the repair work commencing.

4-4.11 STRUCTURAL STEEL

4-4.11.1 General

A. This Section 4-4.11 [*Structural Steel*] of this Schedule sets out the requirements for all structural steel forming part of a Transportation Structure, including minimum requirements for the supply, fabrication, delivery and erection of structural steel for Transportation Structures except that Section 4-4.11.12 [*Structural Steel for Building Structures*] of this Schedule sets out the minimum requirements for the supply, fabrication, delivery and erection of structural steel for Building Structures.

1. Structural steel for Transportation Structures must include piling, steel girders, trusses, diaphragms, bracing, fasteners, splice plates, deck drains, anchor rods, dowels, deck joint assemblies, buffer angles, connector angles, anchor bolt sleeves, curb, barrier and median cover plates, trough plates, pier nose plates, pier bracing, bridge rails and miscellaneous steel components and associated materials.

4-4.11.2 Supply and Fabrication Standards

A. The fabricator of structural steel for Transportation Structures must operate a steel fabricating shop that is fully approved by the CWB in accordance with CAN/CSA W47.1 in the following divisions:

1. fabrication of steel girders, girder components, welded steel trusses or other primary load carrying members – Division One;

2. all other Structure components – Division One or Two; and

3. field welding/repairs – Division One or Two.

B. Fabrication of structural steel, including welding, cutting and preparation, must comply with the AASHTO LRFD BCS and the AWS Bridge Welding Code D1.5M/D1.5.

C. The fabrication of structural steel tubing must comply with the AWS Structural Welding Code D1.1/D1.1M.

D. Fabricators of steel girders, girder components, welded steel trusses, and other primary load carrying members must be certified by the Canadian Institute of Steel Construction as meeting the quality compliance requirements in the category of steel bridges.

4-4.11.3 Engineering Data

4-4.11.3.1 Shop Drawings

A. Shop drawings showing all fabrication details must be prepared prior to fabrication and submitted to the City. The shop drawings must include the following:

1. all contractor designed elements, including connections not shown on the applicable Final Design Date, which must be authenticated by a Professional Engineer;

2. all dimensions, which must be correct at 20°C unless otherwise specified;

3. weld procedure identification, which must be shown in the tail of the weld symbols;

4. material properties, material reference standards, and product data sheets for each component;

5. all material splice locations;

6. match markings of splice plates and girders;

7. bearings, which must be centred at -5°C; and

8. camber and splice joint offsets measured to the top of top flange at a maximum spacing of 4 m.

B. Sizes of hardware, shear stud connectors, and any other material shown on the shop drawings must be in the actual units (imperial or metric) of the material being supplied.

4-4.11.3.2 Welding Procedures

A. Welding procedures must be prepared prior to welding for each type of weld proposed.

1. The welding procedures must bear the approval of the CWB.

4-4.11.3.3 Fabrication Sequence

A. Prior to commencement of fabrication, an outline of the fabrication sequence and details of equipment must be prepared.

1. The fabrication scheme must include the order of fabrication and assembly of all the component parts, as well as shop assembly, identification of witness points, and details for surface preparation and coating.

4-4.11.3.4 Mill Certificates

A. Mill certificates are required for all structural steel elements described in Section 4-4.11.1 [*General*] of this Schedule.

B. Mill certificate data and results of impact tests must be obtained prior to shipment of material from the mill.

C. Mill certificates must be obtained for all material before fabrication commences.

D. Mill certificates must be in English.

E. Mill test reports originating outside of Canada or the United States of America must be verified by testing the material to the specified material standards, including boron content, by a laboratory that is certified in Canada in accordance with Schedule 9, Section 6.3 [*Accreditation Standards*].

1. Samples for testing must be collected by personnel employed by the certified laboratory.

2. A verification letter must be prepared by the certified laboratory that includes at a minimum, the applicable mill test reports, testing standards, date of verification testing, and declaration of material compliance.

3. The verification letter must be signed by an authorized officer of the certified laboratory.

4. The boron content must not exceed 0.0008%.

4-4.11.4 Materials

4-4.11.4.1 Structural Steel

A. Structural steel must conform to the standard specified on the applicable final Design Data.

   1. Interpretation of equivalent steels must be in accordance with Appendix “A” of CAN/CSA G40.21 (1976 only).

B. The silicon content for exposed galvanized steel must be as follows.

   1. For structural tubing the silicon content must be less than 0.04%.

   2. For structural sections and plates the silicon content must be less than 0.04% or between 0.15% to 0.25%.

   3. Repair of steel plates or rolled shapes by welding at the producing mill is not permitted.

4-4.11.4.2 Bolts

A. Bolts, nuts and washers must be marked as follows.

   1. Metric bolts must be marked with the symbol A325M and those of “weathering” steel must have the A325M symbol underlined.

   2. Metric nuts must be marked with three circumferential lines with an “M” between two of them or must be marked with a “3” if made of a weathering grade.

   3. Washers must be identified as metric by having an “M” indented in the surface or a “3” for weathering grades.

B. Rotational capacity testing and reporting must be performed in accordance with ASTM F3125/F3125M.

4-4.11.4.3 Stud Shear Connectors

A. All stud shear connectors must comply with the chemical requirements of ASTM A108, Grades 1015, 1018 or 1020. In addition, they must meet the mechanical properties specified in AWS D1.5M/D1.5, Table 7.1 for Type B studs.

4-4.11.5 Welding

A. The deposited weld metal must provide strength, durability, impact toughness and corrosion resistance equivalent to the base metal.

B. Low hydrogen fillers, fluxes and low hydrogen welding practices must be used throughout.

   1. Low hydrogen coverings and fluxes must be protected and stored as specified by AWS D1.5M/D1.5.

4-4.11.5.1 Submerged Arc Welding (SAW)

A. The submerged arc welding process is permitted for all flat and horizontal position welds.

B. All flange and web groove welds must be welded by a semi or fully automatic submerged arc welding process.

C. All web to flange welds and all longitudinal stiffener to web welds must be made by a fully automatic submerged arc welding process.

D. All electrodes and fluxes must conform to the diffusible hydrogen requirements of AWS D1.5M/D1.5 filler metal hydrogen designator H8 or lower. Use of cored filler wires in the submerged arc welding process or shielding gas process will not be permitted.

4-4.11.5.2 Shielded Metal Arc Welding (SMAW)

A. The shielded metal arc welding process is only permitted for girder vertical stiffener to flange fillet welds and for miscellaneous steel components such as deck drains, deck joint assemblies, bridge bearings, pier nose plates, buffer angles and pile splices.

B. SMAW electrodes must conform to the diffusible hydrogen requirements of AWS D1.5M/D1.5 filler metal hydrogen designator H4.

4-4.11.5.3 Metal Core Arc Welding (MCAW) and Fluxed Core Welding

A. The metal core arc welding process utilizing low hydrogen consumables with AWS designation of H4 is only permitted for girder vertical stiffeners and horizontal gussets and miscellaneous steel components such as deck drains, deck joint assemblies, bridge bearings, hand rails, bridgerails and buffer angles.

B. Field application of the metal core arc welding process is not permitted.

C. All electrodes must conform to the diffusible hydrogen requirements of AWS D1.5M/D1.5 filler metal hydrogen designator H4.

4-4.11.5.4 Cleaning Prior to Welding

A. Weld areas must be clean, free of mill scale, dirt, grease and other contaminants prior to welding.

B. For multi-pass welds, previously deposited weld slag must also be thoroughly cleaned prior to depositing subsequent passes.

4-4.11.5.5 Tack and Temporary Welds

A. Tack and temporary welds must not be allowed unless they are incorporated in the final weld.

1. Tack welds, where allowed, must be of a minimum length of four times the nominal size of the weld. The length must not exceed 15 times the weld size and must be subject to the same quality requirements as the final welds.
2. Tack welds must be sufficiently ground out prior to final welding in order for the final weld to have a uniform weld bead.
3. Cracked tack welds must be completely removed prior to welding over.

4-4.11.5.6 Run-off Tabs

A. Run-off tabs must be used at the ends of all welds that terminate at the edge of a member.

1. The tabs must be a minimum of 100 mm long.
2. The tabs thickness and shape must replicate the joint detail being welded.
3. The tabs must be tack welded only to that portion of the material that will not remain a part of the Structure, or where the tack weld will be welded over and fused into the final joint.

4. Tabs must be removed by flame cutting after welding.

4-4.11.5.7 Backing Bars

A. The separation of the faying surfaces between backing bars and material to be welded must not exceed 1 mm.

B. The weld must be 100% fused into the backing bar including at the corners of HSS members.

4-4.11.5.8 Welding at Stiffener Ends

A. Stiffeners and attachments fillet welded to structural members must have the fillet welds terminate 10 mm short of edges.

4-4.11.5.9 Preheat and Interpass Temperatures

A. Preheat and interpass temperature requirements must be performed and maintained in accordance with AWS D1.5M/D1.5, except for the following.

   1. All welds on girder flanges and post to base plate groove welds must be preheated to a minimum temperature of 100°C unless a higher temperature is required by AWS D1.5M/D1.5 for the flange thickness.

   2. All post to base plate fillet welds must be preheated to a minimum temperature of 60°C.

   3. The preheat temperature of the web to flange joint must be measured 75 mm from the point of welding on the side of the flange opposite to the side where the weld is being applied.

4-4.11.5.10 Methods of Weld Repair

A. Repair procedures for damaged base metal and unsatisfactory welds must be prepared and authenticated by a Professional Engineer prior to repair work commencing.

4-4.11.5.11 Arc Strikes

A. Arc strikes are not permitted.

   1. In the event of accidental arc strikes a repair procedure must be prepared and authenticated by a Professional Engineer.

   2. The repair procedure must include the complete grinding out of the crater produced by the arc strike.

   3. The repair procedure must also include MPI and hardness testing of the affected area. Hardness of the repaired area must conform to the requirements of Section 4-4.11.9.6 [*Hardness Tests*] of this Schedule.

4-4.11.5.12 Grinding of Welds

A. Flange groove welds must be ground flush or to a slope not exceeding 1 in 10 on both sides.

B. Web butt joints and post to baseplate groove welds that meet the profile requirements of AWS D1.5M/D1.5 must not require grinding.

C. Fillet welds must be continuous with uniform size and profile.

   1. Fillet welds not conforming to an acceptable profile as defined in AWS D1.5/MD1.5 must be ground to the proper profile without substantial removal of the base metal.

2. Grinding must be smooth and parallel to the line of stress.

3. Caution must be exercised to prevent over grinding. Over grinding that results in reduced thickness of the base metal or size of the weld is not permitted.

4-4.11.5.13 Plug and Slot Welds

A. Plug welds or slot welds are not permitted.

4-4.11.5.14 Welding to Girder Flanges and Webs

A. With the exception of longitudinal web to flange welds, all stiffeners, gusset plates, or any other detail material welded to girder flanges must be a minimum of 300 mm from any flange groove weld.

B. Shear stud connectors and bolt holes must not be placed within 50 mm of any flange groove weld.

C. With the exception of longitudinal web to flange welds and longitudinal stiffener to web welds, all stiffeners, gusset plates and any other detail materials welded to girder webs must be a minimum of 300 mm from any web groove welds.

4-4.11.5.15 Field Welding

A. SMAW must be used for field welding.

B. Structural field welds are welds that are required to maintain the integrity of the Structure.

C. Field welding of primary load carrying members is not permitted, unless otherwise specified.

D. All material to be field welded must be prepared in the shop.

E. Where structural field welds are carried out, the following requirements must be met:

1. all welding, cutting and preparation must comply with the AWS D1.5M/D1.5;

2. only welders approved by the CWB in the particular weld category to be carried out must perform weldments;

3. welding procedures approved by the CWB must be prepared for the welds;

4. low hydrogen fillers, fluxes and welding practices must be used in accordance with Section 4-4.11.5 [*Welding*] of this Schedule;

5. when the air temperature is below 10°C, all materials to be welded must be preheated to 100°C for a distance of 80 mm beyond the weld and must be sheltered from the wind;

6. when the air temperature is below 0°C, welding is not permitted unless suitable hoarding and heating is provided. The air temperature inside the enclosure must be a minimum of 10°C. If the steel temperature is less than 10°C, all materials to be welded must be preheated to 100°C for a distance of 80 mm beyond the weld and must be sheltered from the wind; and

7. all structural field welds must be visually inspected by an independent welding inspector certified to Level Three of CAN/CSA W178.2.

F. Where non-structural field welds are carried out, the following requirements must be met:

1. journeyman welders with Class B tickets must perform weldments;

2. welding procedures must be prepared and authenticated by a Professional Engineer;

3. low hydrogen fillers, fluxes and welding practices must be used in accordance with Section 4-4.11.5 [*Welding*] of this Schedule;
4. when the air temperature is below 5°C, all materials to be welded must be preheated to 100°C for a distance of 80 mm beyond the weld and must be sheltered from the wind; and
5. when the air temperature is below 0°C, welding is not permitted unless suitable hoarding and heating is provided. The air temperature inside the enclosure must be a minimum of 10°C. If the steel temperature is less than 10°C all materials to be welded must be preheated to 100°C for a distance of 80 mm beyond the weld and must be sheltered from the wind.

4-4.11.6 Fabrication

A. The fabrication of structural steel components must be carried out so as to not adversely affect the performance of the steel including its strength, durability, impact toughness and corrosion protection.

B. Fabrication must be performed in a fully enclosed area which is heated to at least 10°C.

C. Only welders, welding operators and tackers approved by the CWB in the particular weld category to be carried out must be permitted to perform weldments.

4-4.11.6.1 Cutting of Plate

A. All plate material for main members, such as girders, trusses, splice plates and any plate material welded to main members must be flame cut using an automatic cutting machine.

   1. Shearing is not permitted.

B. All flange material must be cut so that the direction of the applied stress will be parallel to the direction of the plate rolling.

C. As plate material is subdivided for main members, all heat numbers must be transferred to each individual plate.

   1. The numbers must remain legible until such time as the material location in the final assembly has been recorded.
   2. Mill identification numbers stamped into the material must be removed by grinding.
   3. Steel stamps must not be used. The only exception is the match marking of splice plates which may be steel stamped using low stress stamps.
   4. The stamps and specific locations of such stamps must be shown on the shop drawings.

4-4.11.6.2 Flame Cut Edges

A. The flame cut edges of flanges must have a maximum Brinell hardness as stated by Section 4-4.11.9.6 [*Hardness Tests*] of this Schedule.

   1. The surface roughness of the flame cut edge must not be greater than ANSI B46.1 500 μin. (12.5 μm) and be such as to allow Brinell hardness testing without spot grinding.
   2. Brinell hardness tests must be performed at random on the as is flame cut edges. If the hardness exceeds the requirements, the edges must be repaired so that they meet the requirements.

B. All blow backs, signs of lamination, or any other discontinuity detected on plate cut edges for tension members observed during the cutting of the material must be documented.

1. The extent of the lamination must be determined by an ultrasonic testing technician certified to Level II of CGSB and employed by a CAN/CSA W178.1 certified non-destructive testing company.

2. A report and repair procedure must be prepared and authenticated by a Professional Engineer indicating whether or not the material is suitable for fabrication.

4-4.11.6.3 Corner Chamfers

A. Corners of all flanges must be ground to a 2 mm chamfer.

B. Corners of stiffeners, structural sections and plates must be ground to a 1 mm chamfer.

4-4.11.6.4 Vertical Alignment

A. The Structure must be fabricated to account for member deflections and to conform to the lines and grades shown in the applicable final Design Data.

   1. For rolled shapes, advantage must be taken of mill camber that may be inherent in the material.

4-4.11.6.5 Shop Assembly

A. General

   1. Primary load carrying members including box girders must be preassembled in accordance with AASHTO LRFD BCS.

B. Plate Girders

   1. The preassembly of plate girders must only require two sections to be assembled at one time.

   2. The detailed method of assembly, including points of support, dimensional checks, method of trimming to length, drilling and marking of splices, must be to the procedure prepared in accordance with Section 4-4.11.3.3 [*Fabrication Sequence*] of this Schedule.

   3. Each individual girder section must meet the camber requirements for that particular length, with the splices between these sections falling on the theoretical camber line for the entire span.

   4. Corrections for variation in flange thickness must be made.

   5. Camber for plate girders must be measured on the top of the top flange. The camber of plate girders must be measured in the “no load” condition.

   6. The camber of each individual girder section must be known for the next two girder sections in the girder line prior to shop assembly of any particular girder section, to allow the use of a best fit line to reduce the effect of any camber differences.

   7. When the camber of the girder fails to meet the required tolerance, a method of repair must be developed and authenticated by a Professional Engineer prior to commencement of repair.

C. Box Girders

   1. The shop assembly of box girders must be the same as for plate girders with the additional requirements specified in this section.

   2. The camber of box girders must be measured on the top of the top flange, and each top flange of a box must individually meet the required camber.

3. Girder sections assembled for splicing must be supported within 2 m of the end of each section. Girder sections must be supported in such a manner as to provide the correct angular relationship at the splice between girder sections while the splices are being reamed or drilled.

4. Shop drawings must clearly indicate the expected dead load deflection of each section and the elevations of the sections while supported for the drilling or reaming of each splice.

4-4.11.6.6 Drilling

A. All splices must be drilled from solid material while assembled or must be sub-punched or sub-drilled and then reamed to full size while in the shop assembly position.

   1. Drilling or reaming must not take place until after shop assembly has been satisfactorily completed.

4-4.11.6.7 Splice Plates

A. After shop assembly, splice plates and attached members must be clearly match marked to ensure proper orientation and location of splice material for erection. All holes must align with holes in the attached members.

   1. The match marking system must be shown on the shop drawings.

B. After shop assembly and match marking, splice plates must be removed, de-burred, solvent cleaned to remove all oil and sandblasted to remove all mill scale to ensure a proper faying surface.

   1. Splice plates must be securely ship-bolted to the girders.

C. At field splice locations, the gap between adjacent girder ends must be 10 mm ± 5 mm.

4-4.11.6.8 Bolt Holes

A. Section 11.4.8 in the AASHTO LRFD BCS must apply except that all bolt holes in load carrying segments of main members and any material welded to main members must be drilled full size or sub-punched 5 mm smaller and reamed to full size.

B. Punching of full size holes for secondary members such as bracings which are not welded to main members must only be allowed for material less than 16 mm thick.

C. Diaphragm bracing members for curved girder bridges are considered primary structural members and therefore punching of full size holes is not permitted.

D. All holes in girder splices and structural members must be circular and perpendicular to the member. Cutting of slotted holes must be done by plasma arc cutting. Holes must be deburred inside and outside and free of nicks and gouges.

4-4.11.6.9 Flame Straightening and Heat Curving

A. Flame straightening and heat curving is not permitted on any material or member except in accordance with a repair procedure prepared and authenticated by a Professional Engineer. The repair procedure must address locations, temperatures and cooling rates.

4-4.11.6.10 Stress Relieving

A. When stress relieving is specified in the applicable final Design Data, it must be performed in accordance with AWS D1.5M/D1.5.

4-4.11.6.11 Handling and Storage

A. All lifting and handling must be carried out using devices that do not mark, damage, or distort the assemblies or members in any way.

B. Girders must be stored upright, supported on sufficient skids and safely shored to maintain the proper section without buckling, twisting or in any way damaging or misaligning the material.

C. Long members, such as deck joint assemblies, buffer angles, columns and chords must be placed on blocking to prevent damage.

4-4.11.6.12 Barrier

A. All barrier rail splices must be completed using properly fitted backing bars.

B. All barrier rail splices must be ground smooth and flush.

C. Rail and post sections must be orientated such that the tube seams are always located at the bottom, except for rectangular tube sections which must have the tube seams orientated towards the bottom or the outside of the barrier.

D. Barrier rail sleeves for field splices and expansion joints must be square and be properly aligned in the rail end. Corners of the sleeves must be rounded and smooth to ensure a good fit.

4-4.11.7 Dimensional Tolerances

4-4.11.7.1 General

A. The normal tolerance for structural steel fabrication and fitting between whole groups must be ± 3 mm unless otherwise specified.

B. The dimensional tolerances for structural members must comply with AWS D1.5M/D1.5, Section 3.5, unless otherwise specified.

   1. Tolerances for box girder camber, sweep and depth must be measured relative to two imaginary surfaces: a vertical plane passing through the centre line of the girder and a surface located at the theoretical underside of the top flanges following the theoretical camber of the girder.

4-4.11.7.2 Girder Camber

A. Camber of beams and girders must be uniform, true and accurate to the centerline of the top flange.

B. Permissible variation in camber must be within ± $(0.2L_t + 3)$ mm; where L_t is the test length in m.

   1. This applies to fabricated pieces only, prior to shop assembly.

   2. During shop assembly, splice points must be located on the theoretical camber line or at a specified amount from the line.

C. Where field splices are eliminated by combining girder segments into longer girder lengths, the cambers of the girders at the eliminated splice points must be within ± 3 mm of the theoretical camber line or a specified amount from the line.

4-4.11.7.3 Combined Warpage and Tilt

A. Combined warpage and tilt of flanges at any cross section of welded I-shape girders or beams must not exceed 1/200 of the total width of the flange or 3 mm whichever is greater at bolted splice locations.

1. Combined warpage and tilt must be determined by measuring the offset at the toe of the flange from a line normal to the plane of the web through the intersection of the centerline of the web with the outside surface of the flange plate.

4-4.11.7.4 Web Panning

A. The maximum variation from flatness for webs must be 0.01*d* where *d* is the least dimension of the panel formed by the girder flanges and stiffeners.

   1. Should the panning in one panel be convex and the panning in the adjacent panel be concave then the sum of the panning in the two adjacent sections must not exceed that allowed for one panel.

   2. If a web exhibits double curvature between the flanges within a single panel, the panning value specific to each direction of curvature must be added together when determining the variation from flatness of the panel.

B. Localized deformations in the web must not exceed 3 mm in 1 m.

4-4.11.7.5 Splices

A. The tolerance for girder depth or box girder geometry must be as specified by AWS D1.5, except that the difference between similar dimensions of adjoining sections being spliced must not exceed ± 3 mm.

B. Bolted splices of main stress carrying members must have parallel planes and the surfaces must be in full contact without any gap.

C. At field splice locations, the gap between adjacent girder ends must be 10 mm ± 5 mm.

D. Filler plates are not permitted at main girder field splices unless specified.

4-4.11.7.6 Stiffeners

A. The bearing ends of bearing and jacking stiffeners must be flush and square with the web and must have at least 75% of the bearing end area in contact with the flanges.

B. Tolerance for milled to bear stiffeners must be 0.05 mm with at least 75% of the area in bearing.

C. Fitted stiffeners may have a gap of up to 1 mm between the stiffener and the flange.

4-4.11.7.7 Bearing to Bearing Dimension

A. The bearing to bearing distance must not exceed ± 2 mm.

4-4.11.7.8 Facing of Flanges

A. Surfaces of flanges which are in contact with bearing sole plates must have a flatness tolerance of 0.001 x bearing dimension.

4-4.11.7.9 Deck Joint Assemblies

A. Tolerances for straightness must be accounted for over the length of the assembly between the crown and gutter line both before and after galvanizing of the assembly.

   1. Deviation from straightness in a vertical plane must not exceed ± 6 mm.

   2. Horizontal sweep or variations in gap setting must not be greater than ± 3 mm.

3. Deck joint assemblies must be assembled for inspection in a relaxed condition with erection angles removed.

4-4.11.7.10 Barriers

A. Individual barrier rail sections must be straight and true with no evidence of kinks or dents and with a minimum variation from straightness not exceeding 3 mm over a 3 m length.

B. Welded barrier rail splices must not be evident in the final product, and must be straight, kink free and conform to the same section as the adjacent section.

C. Bolted barrier rail must be straight with no offset due to loose fitting sleeves.

D. The clearance between barrier rail sections and their sleeves must be sufficient to ensure an easy fit after galvanizing.

E. The maximum radial clearance around the sleeve when fitted into the rail section must be 1 mm (2 mm total) after galvanizing with the tube seam removed.

4-4.11.7.11 Anchor Bolts

A. The bolts in an anchor bolt assembly must fit in a template comprised of accurately located holes 2 mm greater in diameter than the anchor bolts.

B. The top of the bolts in the assembly must be ± 3 mm from a level plane when the threaded portion is plumb.

C. The threaded length must not be less than specified, nor more than 15 mm greater than that specified.

D. The threaded ends of anchor bolts must be chamfered.

4-4.11.8 Surface Preparation and Coating

4-4.11.8.1 Blast Cleaning

A. Unless otherwise specified in the applicable final Design Data, all steel components must be blast cleaned after fabrication in accordance with the SSPC No. SP6.

    1. Essentially this is a surface from which all oil, grease, dirt, rust, foreign matter, mill scale and old paint have been completely removed except for slight shadows, streaks or discolorations caused by rust stain or mill scale oxide binder.

    2. The exterior faces of exterior girders must be uniform in appearance.

4-4.11.8.2 Galvanizing

A. Galvanizing of steel must be by the hot-dip method, after fabrication, in accordance with ASTM A123/A123M Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products and ASTM F2329/F2329M Standard Specification for Zinc Coating Hot-Dip Requirements for Application to Carbon and Alloy Steel Bolts, Screws, Washers, Nuts and Special Threaded Fasteners.

    1. A smooth finish must be provided on all edges and surfaces, and all weld spatter and welding flux residue must be removed from steel components prior to galvanizing.

    2. Nuts must freely spin on bolt threads after galvanizing.

3. Factors contributing to galvanization-induced cracking must be minimized.

B. Exposed galvanized surfaces must have a continuous outer free zinc layer without any significant zinc-iron alloy showing through the outside surface.

1. Lumps, globules or heavy deposits of zinc are not permitted.

2. Members must be free of any sharp protrusions or edges.

C. Galvanizing repairs must provide a coating that has a minimum thickness of 180 µm, adheres to the member and has a finished appearance similar to that of the adjacent galvanizing.

1. Galvanizing repair must comply with ASTM A780A780M, Method A3 “Metallizing” unless the repair area is less than 100 mm^2 in which case the repairs may comply with ASTM A780/A780M Method A1 “Repair Using Zine-Based Alloy”.

2. Galvanizing repairs must be tested for adhesion.

3. Repairs may require complete removal of the galvanized coating and re-galvanizing.

D. The cleaning and pickling procedure of high strength ASTM A194/A193 Grade B7 anchor rods, or any other high strength rod with tensile strength exceeding 830 MPa, must be modified as follows prior to hot-dip galvanizing:

1. brush blast to remove mill scale and oil after threading ends;

2. flash pickle up to 5 minutes; and

3. quick dry prior to hot-dip galvanizing (not stored in flux or acid rinse).

4-4.11.8.3 Base Plate Corrosion Protection

A. The bottom surface of galvanized base plates in contact with concrete must be protected by a medium grey colour barrier coating to prevent contact between the zinc and the concrete.

1. The galvanized surface must be roughened prior to application of the barrier coating.

2. The surface preparation of the galvanized surface and the dry film thickness of the coating must be in accordance with the coating manufacturer’s recommendations.

3. The adhesion of the fully cured coating must be tested in accordance with ASTM D3359. The method selected for testing (Method A or B) must depend on the dry film thickness of the coating. The adhesion test result must meet a minimum of “4B” classification.

4-4.11.8.4 Prime Coating

A. At bearing locations, a prime coat must be applied to the underside of bottom flanges in contact with the bearing sole plate. The primer must extend the full width of the flange and 15 mm beyond the projected contact surface of the bearing sole plate in the longitudinal direction.

B. At all deck joint locations, a complete SF2, SF3 or SF4 approved bridge coating system from the Alberta Transportation Products List must be applied to the bottom flange surfaces (underside, top and edges), with the exception that the faying surface of the underside of the bottom flange in contact with the bearing sole plate must only receive the prime coat.

1. The coating system must extend longitudinally from the girder end to a distance 100 mm beyond the bearing sole plate or 100 mm beyond the jacking stiffener, whichever is greater.

2. The selected SF2, SF3, or SF4 coating system must be applied to the full height of the bridge webs (both sides of web and including any applicable bearing/jacking stiffeners) and to the underside of the top flanges. The longitudinal extent of this coating must be the same as described in Section 4-11.8.4.B.1 [*Prime Coating*] of this Schedule.

3. Faying surfaces of bolted connections must only receive the prime coat.

C. Any portions of the girder that will be encased in cast-in-place concrete must be left in bare steel condition with no prime coat applied.

D. The prime coat must be an organic zinc epoxy primer that has been qualified by test as a Class B coating, in accordance with the "Testing Method to Determine the Slip Coefficient for Coatings Used in Bolted Joints" as described in Appendix A of the Research Council on Structural Connections "Specification for Structural Joints Using High Strength Bolts".

4-4.11.9 Inspection and Testing

4-4.11.9.1 General

A. Inspection and testing must be carried out as required in this Section to ensure that the structural steel, including all welds, has the required properties.

B. Quality control inspection and testing must be completed by the Design-Builder. Quality control must be the sole responsibility of the Design-Builder.

C. Quality assurance inspection and testing completed by the Designer and/or City must not be considered as relieving the Design-Builder of their sole responsibility for completing the Project Work in accordance with the Agreement.

D. Testing and inspection must comply with the following standards:

1. Radiography – AWS D1.5M/D1.5

2. Ultrasonic – AWS D1.5N/D1.5

3. Magnetic Particle – ASTM E709

4. Dye Penetrant – ASTM E165/E165M

5. Hardness Tests – ASTM A833 for cut flange plate and ASTM A1038 for accidental arc strike

E. Visual inspections of welds must be carried out by an independent welding inspector certified to Level 3 of CAN/CSA W178.2.

F. Non-destructive testing must be carried out by a company certified to CAN/CSA W178.1.

1. Radiographic testing and magnetic particle testing technicians must be certified to Level II of CGSB.

G. Full access for the inspection of material and workmanship must be provided to the City.

1. When required by the City, the Design-Builder must provide needed manpower for assistance in inspection duties.

4-4.11.9.2 Radiographic Inspection of Girders

A. Unless otherwise specified, radiographic inspection of girders must be performed in accordance with the following schedule:

1. 100% of all tension and stress reversal flange groove welds, all stiffener butt welds and all diaphragm groove welds, and any groove welded attachments to flange plates;
2. a minimum of 25% of all other flange groove welds randomly selected for each Transportation Structure;
3. 100% of all web groove welds; and
4. additional testing must be carried out if required to ensure the quality of welds.

B. In addition to radiographic inspection, for steel members with a thickness of 65 mm or greater, 25% of the welds must be supplemented by ultrasonic inspection.

C. Deficiencies discovered must be repaired and the suspect area re-inspected.

4-4.11.9.3 Radiographic Inspection of Members Other than Girders

A. Unless otherwise specified, radiographic inspection of groove welds in members other than girders must be performed in accordance with the following schedule:

1. 100% of all tension members;
2. 100% of all barrier rail splices; and
3. 50% of all other members.

B. In addition to radiographic inspection, for steel members with a thickness of 65 mm or greater, 25% of the welds must be supplemented by ultrasonic inspection.

C. 25% of post to base plate welds must be inspected by ultrasonic or radiographic inspection.

D. Deficiencies discovered must be repaired and the suspect area re-inspected.

4-4.11.9.4 Magnetic Particle Inspection

A. Unless otherwise specified, magnetic particle inspection of girders must be performed for each girder section in accordance with the following schedule:

1. 50% of the web to flange welds or any fillet welds placed on flange plates. The tests must be in 1.5 m lengths including a 1.5 m length at each end of the web to flange weld;
2. 20% of the web to stiffener welds;
3. 100% of the stiffener and diaphragm connector plate to flange welds;
4. 100% of the bearing sole plate to flange welds;
5. 20% of the diaphragm connector plate welds to web;
6. 100% of all manual (SMAW) welds; and
7. 25% of all fillet welds for all other components.

B. Deficiencies discovered must be repaired and the suspect area re-inspected.

4-4.11.9.5 Dye Penetrant Inspection

A. Dye penetrant inspection must be performed at the ends of the weld metal of all flange groove welds after the removal of run-off tabs.

B. Dye penetrant inspection must be done for all flange plate edges regardless of whether or not the plates are cut before or after welding.

C. Deficiencies discovered must be repaired and the suspect area re-inspected.

4-4.11.9.6 Hardness Tests

A. Hardness tests must be performed on the flame cut edges of girder flanges prior to assembly.

   1. If grinding is required to obtain a reliable hardness reading, the full length of the plate edge must be ground.

B. A minimum of three readings for each cut edge of the plate (at each end and midpoint along the length of the edge) must be taken.

C. Unless otherwise specified, the hardness of the flame cut edges must not exceed the following maximum Brinell hardness's.

   1. For carbon steels with a yield strength less than and including 300 MPa, the maximum Brinell must be 200 BHN.

   2. For carbon steels with a yield strength greater than 300 MPa, the maximum Brinell must be 220 BHN.

D. Remedial work to the edges which exceed the specified hardness must be performed and the edges re-inspected prior to assembly.

4-4.11.9.7 Testing Stud Shear Connectors

A. Stud shear connectors must meet all requirements as outlined by AWS D1.5M/D1.5.

   1. Bend testing must be performed in accordance with AWS D1.5M/D1.5, in the presence of the Designer.

   2. When bend testing occurs, the studs must be bent towards the centre of the girder.

   3. All remaining studs must be tested by striking with a hammer. A dull sound indicates incomplete fusion and a bend test will then be required for a potentially defective stud.

4-4.11.9.8 Coatings

A. The dry film thickness of the primer, mid-coat and top coat applied to each girder flange, web, stiffener, and bracing members required to be coated as specified in Subsection 4-4.11.8.4, [*Prime Coating*] must be measured. Dry film coating thickness measurements must be completed at a minimum of five locations of each flange, web, stiffener and bracing members in accordance with SSPC-PA 2 Measurement of Dry Coating Thickness with Magnetic Gauges.

4-4.11.9.9 Testing of Deck Joint Strip Seal

A. Upon completion of strip seals and cover plated v-seals deck joint assembly installation, the assemblies must be tested by the Design-Builder in the presence of the Designer for leakage by flooding with water. Any areas of leakage must be investigated and a repair procedure developed by the Design-Builder and submitted to the Designer for review and acceptance. The Design-Builder must not complete any repairs until written acceptance of the repair procedure has been provided by the Designer and the City. Seals with cracks, tears, puncture damage, or other damage must be removed and replaced, as determined by and to the satisfaction of the Designer.

4-4.11.10 Structural Steel Erection

4-4.11.10.1 General

A. Structural steel must be erected in a manner that does not damage or adversely affect the steel. The erection procedure must maintain girder stability; location; and horizontal, vertical and longitudinal alignment at all times.

B. Drilling of additional holes and any other modifications including field welding is not permitted.

C. Lifting forces must be vertical.

D. Lifting devices must not be welded to girders or require the removal of any stud shear connectors.

E. Steel girders must be erected with cranes.

F. Structural steel must not be erected until the substructure concrete has been cured a minimum of 3 days and achieved 80% of the 28-day specified concrete strength requirement.

4-4.11.10.2 Transporting Materials

A. Girders and beams must be transported and stored with webs in the vertical position, unless otherwise specified.

B. Structural steel must be protected from dirt, road salts, slush or other contaminants during transportation, handling, and storage. All girders must be cleaned of all loose or foreign material prior to erection or installation in accordance with Subsection 4-4.11.10.3 [*Cleaning*] of this Schedule.

C. Girders and beams may be transported in other positions provided:

   1. A Professional Engineer prepares an authenticated transportation assessment report and provides a written statement that the transportation method will not damage the elements. The assessment must account for all static and dynamic forces and associated stresses experienced by the girders during handling, transportation and storage including a dynamic load allowance of at least 100%. The maximum cyclic stress range must not exceed the constant amplitude fatigue threshold for the appropriate fatigue categories specified in CAN/CSA S6.

   2. Upon arrival at the Site and prior to erection, the elements must be checked for correct tolerances, material cleanliness and presence of damage.

   3. An adequate flat storage area must be provided for the inspection.

D. Any element damaged during transportation, handling, storage or erection must be immediately reported to the City.

   1. An engineering assessment report and repair procedure prepared and authenticated by a Professional Engineer experienced in the evaluation and inspection of damaged concrete members must be submitted to the City.

   2. An independent inspection and assessment must be performed on the damaged elements by a Level III certified welding inspector in accordance with CSA 178.2 accredited with W47.1.

E. All elements must be lifted and handled using devices that do not mark, mar, damage or distort the elements and assemblies in any way.

F. Galvanized material must be stacked or bundled and stored to prevent wet storage stain in accordance with the AHDGA publication “Wet Storage Stain”.

   1. Galvanized steel exhibiting heavy wet stain must have affected areas repaired according to ASTM A780/A780M prior to installation.

4-4.11.10.3 Cleaning

A. Structural steel that becomes contaminated with dirt, mud, road salts, slush or other contaminants during transportation, handling and storage, must be cleaned to the satisfaction of the Designer and in accordance with the following requirements a minimum of 48 hours prior to erection, assembly or installation.

   1. Cleaning must be completed by high pressure washing and/or steam.

      a. The washing system used for cleaning must maintain wash water at a minimum temperature of 90°C, have a minimum pressure of 3000 PSI, maximum pressure of 3500 PSI, and a minimum flow rate of 5 GPM.

      b. Wash water must be potable and of a quality acceptable to the Designer.

      c. Wash water must be contained and managed in a manner acceptable to the Designer and in compliance with all applicable environmental requirements.

      d. Washing must be completed from the highest elevation of the component in its washing position to the lowest elevation.

   2. Components must be supported on timber blocking to provide a minimum clearance of 900 mm from the ground along the entire length of the component.

   3. Components must be stored to provide a minimum clearance of 1200 mm from each other.

   4. Transportation, handling and storage restraints, and their softeners, must be removed or repositioned to permit complete and full cleaning of the structural steel.

   5. Locations of chains, flange hooks, restraint systems, other tie down devices, and timber blocking must be adjusted to permit cleaning at contact areas.

   6. Splice plates, if secured to the steel girders during transportation, must be removed from the girder or separated from the girder to provide a minimum of 75 mm of clearance between splice plates and girders.

   7. Access equipment and/or specialized telescoping pressure washing equipment must be used to wash structure steel that is 3 m or greater above the ground.

B. Structural steel that has been contaminated with road salts, slush or other contaminants must have their surfaces tested for soluble salts after cleaning in accordance with the Bresle Method, ISO 8502-9, Field method for the conductometric determination of water-soluble salts and ISO 8502-6, Extraction of soluble contaminants for analysis. Testing must be completed at a minimum frequency of one test per 30 m2 of structural steel surface area or three tests per component, whichever is greater up to a maximum of six tests per component.

C. Cleaning must result in structural steel surfaces having less than 7 μg/cm2 of chloride ions. All testing must be carried out by the Design-Builder and the results submitted to the Designer for review and acceptance prior to erection, assembly or installation. Structural steel surfaces that exceed the specified limits for contaminants must be further cleaned and testing repeated until the steel surfaces meet the specified limits.

D. Cleaning of the exterior fascia web and flanges of an exterior steel girder must result in a uniform appearance without streaking and free of pressure washing spray patterning. Additional cyclic wetting, fogging and drying may be required to achieve uniformity of appearance as determined by the Designer.

E. Structural steel must be free of ice prior to erection. Methods to remove ice must be reviewed and accepted by the Designer.

F. All costs associated with cleaning, testing, and removing ice from the surfaces of structural steel surfaces will be considered incidental to the Work and no separate or additional payment will be made.

4-4.11.10.4 Temporary Supporting Structures and Berms

A. Temporary supporting structures and berms for the erection of structural steel must be designed and constructed and maintained for the forces that may come upon such temporary supporting structure.

    1. Drawings for temporary supporting structures and berms must be prepared and authenticated by a Professional Engineer.

4-4.11.10.5 Erection of Structural Steel Girders

A. A detailed erection procedure for the erection of structural steel girders and other primary load carrying members must be provided to the City in advance of the scheduled start of erection. The erection procedure must include all drawings and documents necessary to describe the following:

    1. access to the work, including temporary access berms and/or work bridges;

    2. details of Temporary Work, including:

        a. drawings and methods to be used to ensure the required position and stability of the girders or girder segments prior to placing concrete;

        b. location, elevation, and grade of support bearings;

        c. theoretical top of girder elevations at bearing and splice locations; and

        d. vertical, horizontal, and longitudinal position adjustment mechanisms.

    3. an as constructed survey of substructure elements, including:

        a. location and elevation of all bearing grout pad recesses including anchor rod voids;

        b. shim height required at each bearing location; and

        c. longitudinal and transverse measurements between centreline of bearings of all substructure elements.

    4. Superstructure layout plan, including:

        a. installation details of reference lines and markings of substructure and bearing components used to determine final bearing and girder positions.

    5. theoretical top of girder elevations at substructure bearing locations;

6. type and capacity of cranes and other equipment required for handling and erecting of girders;
7. sequence of operation including positions of cranes, trucks with structural steel girders and Transportation Accommodation;
8. position of cranes relative to substructure elements such as abutment backwalls, with details of load distribution of wheels and outriggers;
9. lifting devices and lifting points;
10. girder stabilization details, methods of maintaining girder location and alignment, and details of blocking for girder and bearings;
11. diaphragm and bracing installation schedule and sequence;
12. bolt tightening sequence;
13. grout pad construction; and
14. details of release of temporary supporting structures.

B. The erection procedure must be prepared and authenticated by a Professional Engineer who must assume full responsibility to ensure that its erection procedure is being followed at all times.

4-4.11.10.6 Erection of Barrier Railings

A. The lines and grades of the barrier railings must be true to that shown on the applicable final Design Data, and not follow any unevenness in the Structure.
    1. It may be necessary to adjust the height of the barrier railing, in order to compensate for normal superstructure variations, and achieve the desired line and grade on the barrier railing.

4-4.11.10.7 Straightening Bent Material

A. Straightening of plates, angles or other shapes must only be permitted based on a detailed procedure authenticated by a Professional Engineer.
    1. Following the straightening of a bend or buckle, the surface of the metal must be inspected for evidence of fractures.
    2. Inspection may include non-destructive testing.

4-4.11.10.8 Assembly

A. The structural steel parts must be assembled as shown on the shop drawings and all matchmarks must be followed.
    1. Hammering is permitted, provided it does not cause any damage to structural steel components, such as:
        a. enlarging holes;
        b. damaging or distorting bolts;
        c. damaging or distorting cylindrical erection pins; or
        d. damaging structural members.

2. Bearing surfaces and surfaces to be in permanent contact must be clean before the members are assembled.

3. Splices and field connections must have 50% of the holes filled with bolts and cylindrical erection pins (half bolts and half pins evenly distributed throughout the splice or connection) before bolting. Splices and connections carrying traffic during erection must have at least 75% of the holes filled.

4. Fitting up bolts must be sized to the same nominal diameter and be distinguishable from the final bolts. Cylindrical erection pins must be sized to accurately fit the holes.

5. Should adjustments in elevation of the girder or primary load carrying member splices become necessary to allow free rotation of the joint, only enough pins or bolts to allow rotation must be removed.

4-4.11.10.9 Grout Pads

A. Grout pads must comply with Sections 4-4.14.10.4 [*Grout Pads*] and 4-4.14.10.5 [*Grouting in Cold Weather*] of this Schedule.

4-4.11.11 High Tensile Strength Bolted Connections

4-4.11.11.1 General

A. All girders and primary load carrying members must be erected with elevations and alignments checked for conformance to the lines and grades shown on the applicable final Design Data prior to any bolt tightening.

B. Bolted parts must fit solidly together when assembled.

1. Contact surfaces, including those adjacent to the washers, must be descaled or carry the normal tight mill scale as required by the final Design Data. The final Design Data must indicate what type of surface preparation has been assumed.

2. Contact surfaces must be free of dirt, paint, oil, loose scale, burrs, pits and other defects that would prevent solid seating of the parts.

C. Unless otherwise specified, bolts in exterior girders must be installed with the heads on the outside face of the girder web and bolts in all girders must be installed with the heads on the bottom faces of lower flanges.

D. Nuts for bolts that will be partially embedded in concrete must be located on the side of the member that will be encased in concrete.

E. Connections must be assembled with a hardened washer under the bolt head or nut, whichever is the element turned in tightening. The smooth side of the hardened washer must be placed against the structural steel.

1. Surfaces of bolted parts in contact with the bolt head and nut must be parallel.

2. For sloped surfaces sloping more than 1:20, ASTM F436 bevelled washers must be used. The bevelled washers must produce a bearing surface normal to the bolt axis.

F. Bolts must be new and stored in weatherproof containers to prevent loss of lubrication or accumulation of dirt. Re-lubrication of bolts before installation is permitted.

4-4.11.11.2 Pre-Installation Verification Testing

A. Prior to erection and the installation of bolts in joints/connections, the Design-Builder must complete pre-installation verification testing in the presence of the Designer of the bolt assembly lot to be installed. Pre-verification testing of the bolt assembly lot must be completed no more than 1 week prior to their installation. Each combination of bolt, nut and washer manufacturing lots will be considered a bolt assembly lot.

B. Pre-installation verification testing of bolts must be completed in accordance with Section 7 of the Research Council on Structural Connections Specification for Structural Joints Using High Strength Bolts and this specification. In the case of conflicting requirements, the requirements of this specification must govern.

C. Each tool to be used in the installation of bolts to complete snugging and tensioning procedures must be used to complete a minimum of one of the pre-installation verification testing samples.

D. Each worker who will be installing bolts must successfully complete one of the pre-installation verification test samples in the presence of the Designer using the same tools and methods that will be used in the actual installation. Any worker who under or overtightens bolts during pre-installation verification testing will not be permitted to install bolts unless they successfully complete a retest in the presence of the Designer. If any bolt fails to meet the specified minimum tension, the entire bolt assembly lot will be rejected.

4-4.11.11.3 Field Rotation Capacity Testing

A. Rotational capacity testing reports of bolt assembly lots must be reviewed and accepted by the Consultant in accordance with Section 4-4.11.4.2 [*Bolts*] prior to the commencement of pre-installation verification testing.

B. If in the opinion of the Designer the condition of bolt assemblies have changed during the Project Work, the Design-Builder must complete field rotational capacity testing. Field rotational capacity testing must be completed in accordance with ASTM F3125/F3125M in the presence of the Designer. If any bolt sample tested fails to meet the specified requirements, as determined by the Designer, the entire bolt assembly lot will be rejected.

4-4.11.11.4 Bolt Tension

A. Tightening of all high strength bolts must be by the turn-of-nut method.

B. Before final tightening there must be a sufficient number of bolts brought to a “snug tight” condition to ensure that the parts of the joint are brought into full contact with each other. Following this initial operation, bolts must be placed in any remaining holes in the connection and brought to snug tightness.

   1. For the purposes of this Section 4-4.11.11.4 [*Bolt Tension*], “snug tight” must mean the tightness attained by a few impacts of an impact wrench or the full effort of a person using an ordinary spud wrench.

C. After all bolts have been taken to the snug tight condition, the outer face of each nut and end of bolt must be match marked to have a common reference line to determine the relative rotation. All bolts in the joint must then be tightened additionally by the applicable amount of nut rotation specified below, with tightening progressing systematically from the most rigid part of the joint to its free edges. During this operation there must be no rotation of the part not turned by the wrench.

D. The amount of rotation of the nut relative to the bolt from snug tight, regardless of which is turned must be:

1. 1/3 turn where the bolt length is four bolt diameters or less;
2. 1/2 turn where the bolt length is over four bolt diameters and not exceeding eight bolt diameters; and
3. 2/3 turns where the bolt length exceeds eight bolt diameters.

E. The rotational tolerance must be 1/6 turn (60°) over and must not be under.

F. The length of the bolt must be measured from the underside of the bolt head.

G. The Field Review Monitor must witness the tightening of 10% of all bolts at the following times:

1. the marking of the nuts after they are snug tight; and
2. after they have been tightened by the turn-of-nut method.

4-4.11.11.5 Reuse of Fasteners

A. High strength bolts must be tensioned only once and must not be reused.

B. Retightening previously tightened bolts, which may have been loosened by tightening adjacent bolts must not be considered as reuse.

4-4.11.11.6 Misfits

A. The correction of minor misfits involving reaming, cold cutting and chipping for secondary members may be permitted by the City, in its discretion.

1. If such field corrections are required, a repair procedure authenticated by a Professional Engineer must be prepared and submitted to the City prior to the corrections being carried out.

4-4.11.11.7 Girder Adjustment

A. Adjustments to girder positions, bearing locations and bearing elevations must be made as required to achieve as closely as practicable the lines and grades shown on the applicable final Design Data.

B. Structural steel must be maintained in correct alignment until the adjoining or encasing concrete components have been completed.

C. Reaming of primary members is not permitted.

4-4.11.12 Structural Steel for Building Structures

4-4.11.12.1 General

A. This Section 4-4.11.12 [*Structural Steel for Building Structures*] sets out the requirements for all structural steel forming part of a Building Structure, including minimum requirements for the supply, fabrication, delivery and erection of structural steel for Building Structures.

1. Structural steel for Building Structures includes metal decking, cold-formed sections, open web steel joists, beams, girders, purlins, wall girts, columns, frames, bracing, bridging, edge angles, respective attachments, plates, bolts, metal studs, metal stair stringers, open grid floor grating, handrails, floor trenches, stair and landing pans and ladders.

4-4.11.12.2 Standards

A. Structural steel must comply with the applicable requirements of the following standards:

1. ASTM F3125/F3125M – High Strength Structural Bolts, Steel and Alloy Steel, Heat Treated, 120 ksi (830 MPa) and 150 ksi (1040 MPa) Minimum Tensile Strength
2. ASTM A653/A653M – Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process
3. ASTM A276/A276M – Standard Specification for Stainless Steel Bars and Shapes
4. ASTM A123/A123M – Standard Specification for Zinc (Hot-Dip Galvanizing) Coatings on Iron and Steel Products
5. ASTM A276/A276M – Standard Practice for Preparation of Zinc (Hot-Dip Galvanized) Coated Iron and Steel Product and Hardware Surfaces for Painting
6. Canadian Institute of Steel Construction/Canadian Paint Manufacturer's Association (CISC/CPMA) CISC/CPMA 1-73b, Quick Drying, One Coat Paint for Use on Structural Steel - Grey
7. CISC/CPMA 2 – Quick Drying, Primer for use on Structural Steel - Grey
8. CAN/CSAG164 M92 – Hot-Dip Galvanizing of Irregularly Shaped Articles
9. CAN/CSA W47.1 – Certification of Companies for Fusion Welding of Steel Structures
10. CAN/CSA W48 – Filler Metals and Allied Materials for Metal Arc Welding
11. CAN/CSA W55.3 – Certification of Companies for Resistance Welding of Steel and Aluminum
12. CAN/CSA W59 – Welded Steel Construction (Metal Arc Welding)
13. The Society for Protective Coatings (SSPC) SSPC SP 3 – Power Tool Cleaning
14. SSPC SP 6/NACE No. 3 – Commercial Blast Cleaning
15. SSPC SP 8 – Pickling

4-4.11.12.3 Engineering Data

A. Shop drawings and design briefs must be authenticated by a Professional Engineer.

   1. Shop drawings must include erection drawings, elevations and details.
   2. Shop drawings must indicate welded connections using welding symbols in compliance with CISC Welding Standards. Net weld lengths must be clearly indicated.

B. All sketches and design calculations for contractor designed elements must be authenticated by a Professional Engineer.

C. For metal deck shop drawings, clearly indicate design loads, material specifications, decking plan, deck profile dimensions and thicknesses, anchorage, supports, projections, openings and reinforcement, closures, flashings, applicable details and accessories.

D. Shop drawings for joists and connections must be authenticated by a Professional Engineer.

E. Structural steel design must comply with the following standards:

   1. CAN/CSA S16 – Design of Steel Structures

2. CAN/CSA S136 – North American Specification for the Design of Cold-formed Steel Structural Members

3. CISC – Code of Standard Practice for Building

4. CISC – Steel Joist Facts

5. CISC – Guide for Specifying Architecturally Exposed Structural Steel

F. Shop drawings, procedures and calculations must be submitted to the City.

4-4.11.12.4 Materials

A. Welding materials must comply with CAN/CSA W48 and W59.

B. Galvanizing must be touched up with a minimum of two coats of zinc rich primer for interior exposures.

4-4.11.12.5 Fabrication

A. Fabrication of structural steel members must comply with CAN/CSA S16 and CAN/CSA S136.

B. Fabrication of metal deck must comply with CAN/CSA S136 and CSSBI Standards and Drawings.

C. All Project Work must be performed by a firm certified by the CWB to the requirements of CAN/CSA W47.1 in Division One or Division Two.

D. All welders employed for erection must possess valid "S" Classification Class "O" certificates issued by the CWB.

E. All gaps, butt joints and connections exposed to the exterior of a Building Structure must be closed and weatherproofed. All exposed welds must be ground flush with surface of welded members.

F. Connections for structural steel must be designed and detailed so that the potential for corrosion is minimized. All exposed ends of HSS sections must be capped and seal welded.

4-4.11.12.6 Inspection & Testing

A. Radiographic and magnetic particle inspection of all full penetration welds and column splices must be performed in accordance with CAN/CSA W59 and ASTM E109.

B. All welds must be visually inspected.

C. Welds must be considered to be Deficiencies if they fail to meet the quality requirements of CAN/CSA W59.

D. High strength bolted connections must be inspected and tested in accordance with Clause 23.9 of CAN/CSA S16.

E. Free access must be provided to the City to all portions of Project Work in the shop and in the field.

4-4.11.12.7 Erection

A. All members damaged during transit or erection must be repaired or replaced.

B. Structural steel must be erected in accordance with CAN/CSA S16 and the applicable final Design Data.

C. Welding must not be carried out at temperatures below 5ºC except with express permission of a Professional Engineer.

D. The requirements of CSA W59 for minimum preheat and interpass temperatures must apply.

E. Erection errors must not exceed the requirements of CAN/CSA S16.

F. Steel joists must be erected in accordance with CAN/CSA S16 and the applicable final Design Data.

G. Metal deck must be erected in accordance with the requirements of CAN/CSA S136 and CSSBI and the applicable final Design Data. The deck must be aligned and leveled on the structural supports.

H. All end joints must be located over supports.

I. The lines of supporting steel must be laid out on the top surface of the deck to produce accurate welds and prevent burns through the deck from improper weld locations.

J. Openings through structural members must be designed and authenticated by a Professional Engineer.

K. Immediately after installation, welds, burned areas and damaged areas of zinc coating must be touched up with zinc rich primer.

L. If two or more adjacent flanges on any deck section are concave or convex so that only the edges or crowns touch a straight edge, the deck sections must be repaired or replaced.

M. A Professional Engineer's written permission must be obtained prior to field cutting or altering steel members.

N. After erection, welds, nuts, bolts and washers must be field primed. Abrasions and damage to shop primed and galvanized surfaces must be touched up with field primer.

4-4.12 PAINTING

4-4.12.1 General

A. This specification is for shop painting of newly fabricated structural steel for bridges. Field painting of new structural steelwork will not be permitted.

B. Where standards and standard specifications are referenced, the version current at the time of tendering must govern, unless a specific date is identified. Metric versions are inferred, when available and relevant.

C. This specification describes requirements for several different methods of preparation and for several different approved coating systems which may be applied to bridge structures. Each painting contract must have special provisions and/or drawings which delineate the applicable area of the structure and the coating system to be applied to it.

4-4.12.2 Reference Standards

A. SSPC Standard Procedure for Evaluating the Qualifications of Painting Contractors, SSPC-QP1

B. SSPC Standard Procedure for Evaluating the Qualifications of Painting Contractors to Remove Hazardous Paint, SSPC-QP2

C. SSPC Guide 6, Guide for Containing Debris Generated During Paint Removal Operations

D. SSPC SP 1 Solvent Cleaning

E. SSPC SP 2 Hand Tool Cleaning

F. SSPC SP 3 Power Tool Cleaning

G. SSPC SP 5 White Metal Blast Cleaning

H. SSPC SP 6 Commercial Blast Cleaning

I. SSPC SP 7 Brush-Off Blast Cleaning

J. SSPC SP 10 Near-White Blast Cleaning

K. SSPC SP WJ-4/NACE WJ-4 - Light Cleaning, Waterjet Cleaning of Metals

L. SSPC AB1 Mineral and Slag Abrasives

M. SSPC AB2 Cleanliness of Recycled Ferrous Metallic Abrasives

N. SSPC AB3 Newly Manufactured or Re-Manufactured Steel Abrasives

O. SSPC-PA Guide No.11 Protecting Edges, Crevices, and Irregular Steel Surfaces by Stripe Coating

P. SSPC-PA 2 Measurement of Dry Coating Thickness with Magnetic Gauges

Q. Alberta Transportation Fish Habitat Manual

R. Alberta Transportation Bridge Load Evaluation Manual

4-4.12.3 Contractor Qualifications

A. One of the following four competency levels will be specified in the painting sub-contracts.

1. CQ1: The Design-Builder or painting subcontractor must have certification in good standing with the SSPC-QP2.
2. CQ2: The Design-Builder or painting subcontractor must have certification in good standing with the SSPC-QP1.
3. CQ3: The Design-Builder or painting subcontractor acceptance will be based on submission of documented experience which should include but not be limited to: the names of owners, projects and dates of previous bridge painting projects where containment and disposal of blasting spoil was implemented, copies of any relevant environmental permits and any citations for failure to comply. A list of qualified personnel responsible for the actual paint removal and application will be required. Once accepted no personnel changes must be made without the Designer's written acceptance. Permission will be granted for the Designer to interview the owners, environmental departments and personnel listed above. Falsifying information in the submission will be grounds for disqualification of the bid.
4. CQ4: No specific pre-qualification requirements.

B. Only Design-Builders having the specified level of competency, at the time of tender closing will be considered acceptable.

4-4.12.4 Materials

4-4.12.4.1 Supply

A. The Design-Builder or painting subcontractor must supply all materials to satisfactorily complete the Project Work.

4-4.12.4.2 Blasting Media

A. Design-Builders may choose the type of abrasive intended for use, taking into consideration the abrasive disposal and worker's health implications of each type. The abrasive selected must comply with the applicable SSPC standard.

B. Blasting grit must be free of corrosion producing contaminates and must be free of any moisture, oils, greases or other elements which will reduce the adhesion of paint coatings. The blast cleaning abrasive used must produce the minimum surface profile required by the paint manufacturer.

C. The use of pre-treatment coatings, blasting media additives or treatment of blasting spoil prior to, or subsequent to, disposal must be reviewed and accepted by the City and Designer.

4-4.12.4.3 Paint

A. The Design-Builder must select an approved product from the Alberta Transportation Products List Bridge Coating Systems – Paint. The paint system must be from the category specified in the Project Requirements. The material data sheets and material safety data sheet of the chosen paint system must be submitted with the Design-Builder's work proposal. A single paint system must be used throughout the entire project unless specified otherwise. The Design-Builder must not change to another paint system once the initial paint system has been applied to any portion of the structure.

B. The paint must be delivered in sealed, original, labelled containers bearing the Manufacturer's name, type of paint, brand name, colour designation, batch number and instructions for mixing and/or reducing.

C. For each batch of paint used on the project, the Design-Builder must have an independent laboratory carry out quality control tests for colour, gloss and formulation. The results must be submitted to the City and Designer for review and acceptance a minimum of 2 weeks prior to the commencement of painting. No paint must be applied to the structure until quality control test results have been reviewed and Accepted by the City and the Designer.

D. Colour testing for the mixed top coat must be carried out in accordance with ASTM D2244 with a CIE 1976 L*A*B, 2 degree observer, and a D65 illuminate.

E. Gloss testing for the mixed top coat must be carried out in accordance with ASTM D523 at 60 degrees.

F. Infra red must be conducted on all individual components of the paint system prior to mixing, to confirm that the formulation conforms to that which was originally approved. A minimum of 32 scans must be taken with a Fourier transform infrared spectrometer between 4000 and 400 wave-numbers (CM-1) using the salt plate sandwich technique. The salt plate may be made from potassium bromide for non-aqueous paints. If the formulation contains water; appropriate, non-watersoluble plates must be used. Plate material must be reported with each individual spectrum. The spectra must be taken of vehicle portion of the coating without the pigment. For single component materials, a representative sample of the material must be centrifuged to remove pigment and then analyzed. For plural component materials, each of the individual components must be centrifuged to remove pigment and then analyzed separately. IR analysis of the mixed components is not required. The IR plots must be completed with transmittance (0 – 100%) on the y-axis and Wave-numbers (4000 – 400; non-linear) on the x-axis. All peaks must be labelled with the corresponding wave-number. The spectra must be taken such that the largest peaks are at 50 – 0%T and the baseline is greater than 80%T. Materials with IR plots indicating a change in formulation from that which was originally qualified will be rejected.

G. Each batch of paint may be subjected to additional testing by the City or the Designer.

H. If requested, the Design-Builder must provide 4 x 250 ml samples of paint from a pail or barrel chosen by the City or the Designer. Samples must be placed in suitable new, clean, metal containers; and sealed to avoid contamination of the paint.

4-4.12.5 Priming and Painting

4-4.12.5.1 Stripe Painting

A. Stripe painting is a process whereby an additional layer of paint is applied to all sharp edges of the structure being painted to increase the thickness of the coating around the sharp edge. The Design-Builder must apply stripe paint along all sharp changes in steel surfaces, including but not limited to, edges of flanges, stiffeners, bracing, plates, bolts, nuts, washers, rivets, plates, and sections with sharp profile. Stripe painting may be applied prior to the prime coat or after the prime coat to aid in preservation of the blast cleaned surface. Drying times and recoat conditions must be compatible with all other coats of the paint system. Paint systems using an intermediate coat must also be stripe painted after each intermediate coat, but not after the top coat. Stripe coats when applied over the primer or intermediate coat must be tinted to contrast the underlying coat.

4-4.12.5.2 Paint Application

A. Paint must be applied in accordance with the Manufacturer’s instructions.

B. The use of thinners will only be permitted when reviewed and accepted by the Designer and City. Thinners will only be considered for use where the Design-Builder has CQ1 or CQ2 qualifications. Thinners must be supplied by the manufacturer and formulated for the coat requiring thinning. The thinner must only be used in accordance with and within the range stated on the manufacturers’ published product data sheet. If no range is stated on the manufacturers published product data sheet, the use of thinners will not be permitted.

C. Paint must not be applied when the air and/or steel temperatures are at or below 4°C, nor when the metal has absorbed sufficient heat (above 50°C) to cause the paint to blister and produce a porous paint film, nor when it is possible the air temperature may drop below 0°C before the paint is dry. Variances from these requirements, due to paint supplier’s recommendations or requirements, require the Designer’s acceptance prior to usage.

D. Paint must not be applied to damp or frosty surfaces, nor applied to surfaces when there is a risk of dew. Painting must not commence unless the dry bulb temperature exceeds the wet bulb temperature by more than 5°C and the ambient temperature is rising.

E. Only the anticipated quantity of paint required for 1 day’s work must be opened on that day. Left over paint must not be exposed to the atmosphere. Any paint that becomes oxidized, thickened, ropy, lumpy or dirty must be disposed of.

F. The paint must be mixed in a manner which will ensure breaking up of all lumps, complete dispersion of settled pigment, and provide a uniform composition. The paint must be agitated often enough during application to keep the pigment in suspension.

G. After further evaluation of our qualification testing, and discussions with our committee, we would accept thinning required for application only under the following conditions:

1. Paint must not remain in spray pots, painter's buckets, etc., overnight. Multi-component paints which have been mixed for the duration of the Manufacturer's recommended pot life must be discarded in a safe manner.

2. Paint must be safely stored in a location which keeps its temperature between 10°C to 25°C.

H. Paint must be applied by spraying, brushing, rolling or a combination of these methods. On all surfaces which are inaccessible for brushes or rollers and where spraying cannot be employed, the paint may be applied with sheepskin mitts specifically manufactured for this purpose.

I. Finish coat paint must not be applied over wet touched up primer.

J. All portions of the paint system must be within the range of film thickness(es) stated on the Alberta Transportation Products List Bridge Coating Systems – Paint. Bolts, rivets, edges of members and other changes in surface contour must also receive the required film thickness(es).

K. To ensure that the proper dry film thickness is obtained, the wet film thickness must be checked at the time the paint is applied. The minimum wet film thickness must be equal to the dry film thickness divided by the percentage (expressed as a decimal) of solids in the paint used, with the result rounded up to the next full mil. Each painter must have his own wet film thickness gauge and complete frequent checks of the paint film as it is applied. Dry film thickness must be determined in accordance with SSPC-PA 2 using a calibrated Type 2 constant pressure probe magnetic gauge.

4-4.12.6 Quality Control

A. To ensure that the Project Work done meets the requirements of this specification, the Design-Builder must have an experienced quality control person solely dedicated to actively monitoring and correcting the work of his employees whenever cleaning, surface preparation and coating application is taking place. The Designer will provide a NACE certified quality assurance inspector to monitor and accept the work. The Design-Builder must provide him and all other representatives of the Designer and the City, at their request, safe free access to all areas of the work in all stages of completion.

B. There must be no application of coating materials until the cleaning, and surface preparation have been inspected and accepted by the Designer. Failure to follow this requirement will necessitate the complete removal, by blast cleaning, of all coating placed over surfaces not inspected and accepted by the Designer. Each coat must be thoroughly dry and the mil thickness of each coat accepted by the Designer prior to applying an additional coat.

4-4.12.7 Repair

A. Areas requiring repair, or field touch-up, must be cleaned of all damaged paint and the system reapplied using all coats typical to the original paint system. Each coat must be thoroughly dry before applying subsequent coats. Each coat of repair or touch-up must lap adjacent coating by a minimum of 25 mm onto feathered/roughened existing coating. The Design-Builder must carry out all repairs to the satisfaction of the Designer at no additional cost to the City.

4-4.12.8 5 Year Bridge Painting Warranty

A. The Design-Builder must warranty the Work against all defects in material and workmanship for a period of 5 years. The warranty period will commence on the date of Construction Completion. The Design-Builder must execute the form entitled, "5 Year Bridge Painting Warranty", a sample copy of which is contained in the document at this link http://www.transportation.alberta.ca/Content/docType246/Production/10bcs22.pdf.

B. During the warranty period, the City will conduct yearly inspections of the coating system. A final inspection of the coating system will be carried out a minimum of 60 days prior to the expiration of the warranty period.

C. Failure of the coating system will determined to have occurred when:

1. de-bonding or failure of adhesion of the coating either to the structural steel or lack of intercoat adhesion;

2. the appearance of any rust stains on the structure due to loss of paint or due to leaking from joints between structural members; and

3. loss of gloss or rapid change of colour of the coating.

D. Damage to the coating due to vehicle impact or snow removal equipment will not constitute failure of the system.

E. Repair under warranty must include, but is not limited to, all permitting, approvals, traffic accommodation, containment systems, labour, materials, equipment, tools and incidentals necessary to restore the coating to a condition acceptable to the City at no cost to the City.

F. Warranty repairs must be completed within 60 days of notification or, in the event this would place the repair work period in winter weather conditions, by June 30 of the following year.

4-4.12.9 Shop Coating of Structural Steel for Bridges

4-4.12.9.1 Fabrication Paint Shop

A. Paint shops or areas of fabrication shops where painting is performed must be well lit, free of dust and drafts and maintained at the correct temperature and relative humidity for the coating being applied.

B. Compressed air for cleaning and painting must be free of moisture and oil contamination.

4-4.12.9.2 Pre-Surface Preparation

A. Surfaces to be coated must free of weld spatter, welding flux and cutting slag. All sharp corners and edges must be lightly ground to a 1 mm chamfer to break the sharp edge and all holes must be free of burrs and cutting chips. Oil and grease must be solvent cleaned to meet the requirements of SSPC-SP 1, prior to blast cleaning in preparation for coating.

4-4.12.9.3 Abrasives

A. Abrasives used in shop cleaning must be free of chlorides and other contaminants which could affect the coating being applied and must produce the anchor pattern required by the coating system.

4-4.12.9.4 Blast Cleaning

A. Unless noted otherwise, all fabricated surfaces must be blast cleaned to meet the requirements of SSPC-SP10, which is a surface free of all visible oil, grease, dust, dirt, mill scale, rust, coating, oxides, corrosion products, and other foreign matter. The surface roughness profile must be in accordance with the manufacturer’s published product data sheet.

B. No coatings must be applied until the Designer has inspected and accepted the cleaned surface. Surfaces which have been coated without acceptance of the prepared surface must have the paint removed by blast cleaning and reviewed and accepted by the Designer prior to reapplication of coatings.

4-4.12.9.5 Masking

A. All areas not to be painted must be masked prior to applying paint. This includes portions of members within 100 mm of field weld locations. Unless noted otherwise, all faying surfaces and within 75 mm of open holes must be masked to prevent application of coating. All clip angles and other detail material must be applied after blast cleaning to assure a cleaned faying surface.

4-4.12.9.6 Paint

A. Unless noted otherwise, shop primer must be an inorganic zinc rich primer from the Alberta Transportation Products List Bridge Coating Systems – Paint.

4-4.12.9.7 Intercoat Cleanliness

A. The initial blast cleaned surface and subsequent coats of paint must be kept free of dust, dry spray, overspray, oil and grease prior to application of subsequent coats or shipping.

4-4.12.9.8 Recoat Time

A. The maximum and minimum recoat time for the coating system being applied must be observed and required conditioning agents or surface roughing between coats must be done.

4-4.12.9.9 Shipping Inspection

A. No product is to be shipped until the City and Designer has inspected and accepted the coating. Material shipped without inspection by the City and Designer, may be inspected at the receiving point with all costs of this inspection charged to the Design-Builder.

B. Painted surfaces will be rejected, if any of the following defects are identified:

   1. runs, sags, holidays or shadowing;

   2. evidence of poor coverage at bolts, plate edges, lap joints, crevices, pockets, corners and re-entrant angles;

   3. surfaces which have been struck, scraped, spotted by rain or otherwise damaged;

   4. surfaces which exhibit an objectionable texture such as orange peel, mud cracking, fish eyes, etc.; and

   5. surfaces with over spray.

4-4.12.9.10 Shipping

A. The coating must be protected from damage during shipping.

4-4.13 ASPHALT CONCRETE PAVEMENT (ACP)

4-4.13.1 General

A. ACP must be installed on the bridge deck, roof slabs and approach slabs for the 111 Street Roadway Bridge Widening.

   1. Asphalt concrete pavement must be mix type 10 mm – High Traffic (10 mm-HT) as specified in the D&CS.

   2. Use of reclaimed asphalt pavement on Transportation Structures is not permitted.

B. 10 mm-HT aggregate must be provided to the gradation band shown on Table 4-4.13.1-1 [*10 mm-HT Gradation Band*].

Table 4-4.13.1-1: 10 mm-HT Gradation Band

Designation	**1**
Class	**10.0**
Application	**10mm-HT**
Sieve Size (µm)	**% Passing by Mass**
12 500	100
10 000	97-100
8 000	70-94
6 300	45-85
5 000	32-75
2 500	23-55
1 250	16-45
630	11-36
315	8-26
160	5-15
80	3-8

C. The asphalt mix design must be provided to the City by a qualified laboratory following the SGC and the Bailey Method of Mix Design as set out in the Asphalt Institute Manual Series No. 2 (MS 2) to the criteria shown on Table 4-4.13.1-2 [*Requirements for 10 mm-HT Mix Design*].

Table 4-4.13.1-2 Requirements for 10 mm-HT Mix Design

Parameter	**Requirement**
Number of Gyrations Gyrations N_{design} Gyrations $N_{maximum}$	 100 160
Density at $N_{maximum}$ (%G_{mm})	98.0 max
Bailey CA-CUW	60 to 85 max Fine Graded Or >95 to 105 max Coarse Graded
Air Voids, % of total mix (virgin mix)	4.0 ± 0.4%
VMA, %	13 min
Voids Filled, %	70 - 80
Tensile Strength Ratio, % (AASHTO T283)[1]	80 min
Minimum Film Thickness[2], mm	7.5 min
APA (mm, 52ºC, 8,000 cycles)	5.0 max

Note 1: Minimum Tensile Strength Ratio must be determined in accordance with AASHTO T283, with optional freeze thaw, at air void content of 7.0 ± 0.5%.

Note 2: Minimum film thickness must be determined in accordance with Appendix 02065.B.

D. The asphalt cement must be 150-200 (A) or PG 58-28.

E. The tack coat must be MC-30.

4-4.13.1.2 ACP Mixing Plant

A. The ACP mixing plant used for the preparation of ACP must conform to the D&CS.

   1. The ACP mixing plant must have a certificate of calibration certifying that the plant has been calibrated to produce a uniform mixture complying with the asphalt mix design.

B. The asphalt tank supplying the ACP mixing plant must be equipped with a heating apparatus capable of producing asphalt temperatures up to but not greater than 155ºC uniformly throughout the entire contents of the tank.

   1. The asphalt temperature must be maintained within ± 10ºC of the specified mixing temperature.

4-4.13.1.3 ACP Transportation Equipment

A. ACP must be transported from the ACP mixing plant to the Site in trucks with smooth metal boxes in good and leak proof condition which have been previously cleaned of all foreign materials and hardened ACP mixture.

   1. Excess truck box lubricants, such as detergent or lime solutions, must not be allowed to contaminate the ACP.

   2. Petroleum based truck box lubricants must not be used.

B. Trucks must be equipped with tarpaulins of suitable material and sufficient size to cover the ACP completely and overhang the sides of the truck box when the truck is fully loaded.

   1. Tarpaulins must be securely fastened on all sides of the truck box.

   2. Tarpaulins must be on the truck box whenever ACP is being transported.

4-4.13.1.4 Pavers

A. Pavers must be self-propelled and operated to maintain the lines and grades shown on the applicable final Design Data. For new construction, bridge structures that have more than three travel lanes or longer than 75 m in length must be paved with two or more pavers operating in simultaneous echelon such that the entire width of the ACP lift is completed at one time.

4-4.13.1.5 Compaction Equipment

A. Self-propelled compaction equipment must be used to obtain the required degree of compaction of the ACP.

B. The compaction capability of the equipment used must equal or exceed the placing rate of the spreading operations and must be capable of obtaining the required compaction before the temperature of the ACP falls below specified levels.

C. Compaction equipment must be of a suitable size, weight and type, such that displacement of the ACP and/or disruption of underlying materials will not occur.

1. Specialized compaction equipment must be used as required to achieve adequate compaction and smoothness in tight corners, such as adjacent to deck joints.

D. Compaction equipment must be in proper mechanical condition and operated such that uniform and complete compaction is obtained throughout the entire width, depth and length of the ACP being constructed.

E. A minimum of two pieces of compaction equipment must be used. They must be rollers of at least 10 tonnes mass, one rubber tired roller and one smooth steel drum type roller.

1. Rollers must be configured to ensure uniform and complete compaction up to the face of barriers, curbs, medians and deck joints.

2. Vibrators on vibratory rollers must not be activated.

F. Rollers provided must leave a smooth, properly finished surface, true to grade and cross section without ruts or other irregularities.

G. Compaction equipment must be equipped with methods of wetting the tires or drums to prevent adhesion or pickup of the ACP.

4-4.13.2 Placement of ACP

4-4.13.2.1 General

A. The Structure must be protected to prevent splatter or staining from asphaltic materials.

B. Placement of the first lift of ACP must commence within 7 days of installation of the deck waterproofing membrane.

4-4.13.2.2 Tack Coat

A. Asphalt tack coat must be applied to the protection board and between lifts of ACP.

1. Tack coat must not be applied to wick drains.

B. The surface to be tacked must be dry and free of loose or deleterious material when the tack is applied.

C. The asphalt tack coat must be applied in a uniform manner at an application rate of 0.5 L/m^2.

1. The ambient air temperature at the time of application must be 5ºC or higher.

4-4.13.3 Spreading and Compaction

4-4.13.3.1 General

A. The ACP mixture must be placed only upon a dry, frost free substrate on which the tack coat has cured, and when the ambient air temperature is 5ºC or higher.

B. Prior to delivery of the ACP mixture, the protection board surface must be cleaned of all loose or foreign material.

C. The ACP mixture must be spread and compacted during daylight hours only, unless artificial light is provided.

D. During spreading and compaction operations, care must be taken at all times to ensure that:

1. the ACP mixture is not wasted over the sides of the Structure or onto adjacent surfaces;
2. the deck waterproofing membrane, curbs, barriers, medians and drains are not damaged; and
3. the Structure including guide posts, guardrails, signs, power conduits or any other roadside installations is not damaged.

E. Immediate repairs must be made to any damage resulting from Construction activities.

4-4.13.3.2 Spreading

A. The ACP mixture must be spread at a temperature sufficient for the specified compaction and finishing of the ACP.

B. The manner of placing the ACP must ensure safe accommodation of traffic, quality control and drainage.

C. The longitudinal and transverse edges of the ACP in each traffic lane must be straight in alignment, uniform, and of the same thickness as the adjoining ACP lift.

  1. The exposed edges of ACP lifts must be protected throughout Construction.

D. Each ACP lift must be placed, finished and compacted for its full width, and then allowed to cool down to 50ºC or colder prior to commencing the subsequent lift.

E. In the placing of successive ACP lifts, the individual ACP mixture spreads must be aligned in a manner such that the longitudinal joints in successive lifts do not coincide.

  1. The lateral distance between the longitudinal joints in successive ACP lifts must be not less than 0.30 m.
  2. The longitudinal joints of the final lift of ACP must not be located within the wheel path areas.

F. All longitudinal and transverse joints in the ACP must be of the vertical butt joint type, well bonded and sealed, and must be finished to provide a continuous, smooth profile across the joints.

G. The surfaces of all ACP lifts must not exhibit evidence of segregation.

4-4.13.3.3 Compaction

A. ACP percent compaction must be expressed in percent of maximum theoretical density. The maximum theoretical density used for determining ACP compaction must be as follows:

  1. maximum theoretical density determined on field sampled ACP mixture, or if not available then; and
  2. maximum theoretical density as reported in the accepted mix design.

B. The compaction process must be monitored using a control strip method. Control strips must be established on each lift of ACP placed.

C. The control strip lift must be compacted using at least the following equipment:

  1. one steel roller weighing not less than 10 t; and
  2. one self-propelled pneumatic roller, ballasted to its maximum capacity, weighing not less than 10 t.

D. Once the ACP mixture has been spread by the paver and the initial pass of the breakdown roller has been done, density measurements for determining the control density will commence at five locations within the control strip area and will continue following repeated passes of the compaction equipment until the apparent maximum density is attained. The average maximum density readings from the five control strip test locations will be the Control Density of the lift.

   1. ACP density measurements must be taken for all remaining mats of the lift using nuclear testing equipment.

   2. Following compaction of the lift, density readings must be taken and recorded at a minimum frequency of one per 10 m of bridge length or 20 m of approach road transitions for each mat placed (minimum two tests per mat). The average of the readings taken by the nuclear gauge must be considered the Average Mat Density.

E. The ACP must be compacted to the densities specified in Section 4-4.13.4 [*ACP Paving*] of this Schedule.

F. When the compaction methods and procedures are not achieving the desired compaction, in the opinion of the City, cores of the top lift of ACP must be taken. The number of cores must be determined by the City. The cores must be tested by the Design-Builder and the results provided to the City as soon as they become available.

   1. Coring of the ACP must be carried out using methods which will not damage the asphalt membrane or protection board.

   2. Core holes must be completely de-watered and dried. A generous application of liquid asphalt must be applied to the bottom and sides of the core hole and allowed to cure. ACP mixture must then be tamped in lifts into the core hole until flush with the surface of the surrounding ACP.

   3. Coring must not be undertaken without the consent of the City, in its discretion.

G. In order to maintain the crown of the deck surface, the compaction equipment must not be operated on or across the crown.

H. Compaction procedures and equipment must be such that displacement of the ACP mixture does not occur.

I. Roller wheels on compaction equipment must be kept slightly moistened by water or oil to prevent picking up the ACP mixture, but an excess of either water or oil is not permitted.

4-4.13.4 ACP Paving

A. The completed ACP wearing surface and all intermediate lifts must be smooth and true to the lines and grades show on the applicable final Design Data.

   1. The finished surface of any lift must have a uniform closed texture and must be free of signs of poor workmanship.

B. The ACP wearing surface must be placed and compacted in two nominal 40 mm thick lifts.

C. The first lift of ACP must be spread by the asphalt paver in the direction of the laps in the protection board.

D. To avoid damaging the deck waterproofing membrane, the paver must not exceed the placing rate or push the delivery trucks.

E. Equipment must perform all turning movements off the deck.

F. Dumping of the ACP mixture onto the protection board ahead of the paver is not permitted.

G. The allowable temperature range for compaction of the ACP lifts on the deck waterproofing membrane must comply with Table 4-4.13.4-1 [*Compaction Temperature Range of ACP Lifts*].

Table 4-4.13.4-1 Compaction Temperature Range of ACP Lifts

Asphalt Grade	Compaction Temperature Range	
	First Lift	**Second Lift**
PG58-28 (A)	MAX. 105ºC	128 – 138ºC

H. The minimum average density of the first ACP lift must be 95% of maximum theoretical density with no individual test less than 93% of maximum theoretical density.

I. The minimum average density of the second lift must be 97% of maximum theoretical density with no individual density less than 95% of maximum theoretical density.

4-4.14 BEARINGS

4-4.14.1 General

A. This Section 4-4.14 [*Bearings*] sets out the requirements for all plain and steel reinforced elastomeric bearings and pot bearings forming part of a Structure, including minimum requirements for the supply, fabrication, delivery and installation of bearings.

4-4.14.2 Engineering Data

4-4.14.2.1 Shop Drawings

A. Shop drawings meeting the requirements of Section 4-4.11.3.1 [*Shop Drawings*] of this Schedule and the following must be prepared and submitted to the City.

1. The shop drawings must identify the bearing and clearly indicate all bearing material properties, dimensions, connection attachments, fasteners and accessories.

2. The shop drawings must show the bearing load capacity at the serviceability and Ultimate Limit States as follows:

    a. maximum vertical permanent and total load;

    b. maximum lateral load and corresponding vertical load; and

    c. maximum rotational capacity about any horizontal axis and about the vertical axis at the centre of the bearing.

3. Shop drawings must be authenticated by a Professional Engineer.

4-4.14.2.2 Welding Procedures

A. Welding procedures for the fabrication and field installation of bearings must be prepared prior to welding for each type of weld used in the bearings.

1. The welding procedures must bear the approval of the Canadian Welding Bureau.

4-4.14.2.3 Mill Certificates and Quality Assurance Test Results

A. Mill certificates and quality assurance test results must be obtained for all materials and fabricated components prior to shipping of the finished bearings from the facility of manufacture.

B. Mill test reports originating from a mill outside of Canada or the United States of America must meet the requirements of Section 4-4.11.3.4 [*Mill Certificates*] of this Schedule.

4-4.14.3 Materials

4-4.14.3.1 Steel

A. The steel for baseplate, keeper bars, pintels, pot plates, piston plates, and shims must conform to the requirements of CSA G40.21 Grade 300W or 350W or ASTM A572/A572M Grade 50. The steel for sole plates and fixed rockers must be in accordance with the Drawings. Re-classification of steel using mill test report or mill test report verification data is not permitted.

B. The steel laminates within laminated elastomeric bearings must be rolled mild steel with minimum yield strength of 230 MPa.

C. The boron content of steel to be welded must not exceed 0.0008%.

4-4.14.3.2 Stainless Steel

A. Stainless steel must conform to the requirements of the AISI Type 304. The chemical and mechanical properties must conform to the requirements of ASTM A240/A240M.

B. Stainless steel sheets must have a minimum thickness of 3.2 mm and finished to a No. 8 mirror (0.2 μm) finish.

C. The boron content of stainless steel to be welded must not exceed 0.0008%.

4-4.14.3.3 Elastomer

A. Except for fully integral abutments and piers, cured elastomeric compounds must be low temperature AASHTO Grade 5 and must meet the physical and low temperature brittleness requirements listed in Table X1 and Section 8.8.4 of AASHTO M251.

B. Cured elastomeric compounds for fully integral abutments and piers must be low temperature AASHTO Grade 3, 4 or 5 and must meet the physical and low temperature brittleness requirements listed in Table X1 and Section 8.8.4 of AASHTO M251.

C. Cured elastomeric compounds must also meet the requirements of ASTM D2240 for low temperature crystallinity increase in hardness at an exposure of -25°C for 168 hours.

4-4.14.3.4 Brass

A. Brass sealing rings for pot bearings must conform to the requirements of ASTM B36/B36M, half-hard.

4-4.14.3.5 PTFE

A. PTFE must be unfilled, 100% virgin polymer conforming to Article 18.8.2.5, Unfilled PTFE Sheet of the AASHTO 2017 LRFD Bridge Construction Specifications including all interim revisions. Material used as the mating surface for guides for lateral restraint may be one of the following.

1. Unfilled PTFE.

2. PTFE filled with up to 15% by mass of glass fibres.

4-4.14.3.6 Lubricant

A. Lubricant for bearings must be silicone grease, effective to -40°C, and must comply with U.S. Department of Defense MIL-S-8660C.

4-4.14.3.7 Adhesives

A. Adhesive for bonding PTFE to metal must be an epoxy resin producing a bond with a minimum peel strength of 4 N/mm, when tested according to ASTM D 429, Method B.

   1. Adhesives must not degrade in the service environment.

4-4.14.4 Base Plate Corrosion Protection

A. Bearing base plate corrosion protection requirements must be in accordance with Section 4-4.11.8.3 [*Base Plate Corrosion Protection*] of this Schedule.

4-4.14.5 Welding

4-4.14.5.1 General

A. The deposited weld metal must provide strength, durability, impact toughness and corrosion resistance equivalent to the base metal.

B. Low hydrogen fillers, fluxes and low hydrogen welding practices must be used throughout.

   1. Low hydrogen coverings and fluxes must be protected and stored as specified by AWS D1.5M/D1.5.

C. All electrodes, electrode/flux combinations, and electrode/shielding gas combinations must be CSA certified.

D. Field welding of sole plates to girders by fluxed core welding is permitted.

4-4.14.5.2 Submerged Arc Welding (SAW)

A. The submerged arc welding process is permitted for all flat and horizontal position welds.

B. All welds must be welded by a semi or fully automatic submerged arc welding process.

C. All fluxes must conform to the diffusible hydrogen requirements of AWS D1.5M/D1.5 filler metal hydrogen designator H8 or lower.

4-4.14.5.3 Shielded Metal Arc Welding (SMAW)

A. SMAW electrodes must conform to the diffusible hydrogen requirements of AWS D1.5M/D1.5 filler metal hydrogen designator H4.

4-4.14.5.4 Metal Core Arc Welding (MCAW) and Fluxed Core Welding

A. Field application of the metal core arc welding process is not permitted.

B. All electrodes must conform to the diffusible hydrogen requirements of AWS D1.5M/D1.5 filler metal hydrogen designator H4.

4-4.14.5.5 Cleaning Prior to Welding

A. Weld areas must be clean, free of mill scale, dirt, grease, and other contaminants prior to welding.

B. For multi-pass welds, previously deposited weld metal must also be thoroughly cleaned prior to depositing subsequent passes.

4-4.14.5.6 Tack and Temporary Welds

A. Tack and temporary welds are not allowed unless they are incorporated in the final weld.

   1. Tack welds, where allowed, must be of a minimum length of four times the nominal size of the weld to a maximum of 15 times the weld size and must be subject to the same quality requirements as the final welds.

B. Cracked tack welds must be completely removed prior to welding over.

C. Tack welds must be sufficiently ground out prior to final weld in order to obtain a uniform weld bead.

4-4.14.5.7 Run-off Tabs

A. Run-off tabs must be used at the ends of all welds that terminate at the edge of a member.

B. The thickness and shape of the tabs must replicate the joint detail being welded and must be a minimum of 100 mm long unless greater length is required for satisfactory Project Work.

C. They must be tack welded only to that portion of the material that will not remain a part of the structure, or where the tack will be welded over and fused into the final joint.

D. After welding, the tabs are to be removed by flame cutting, not by breaking off.

4-4.14.5.8 Preheat and Interpass Temperatures

A. Preheat and interpass temperature requirements must be in accordance with Section 4-4.11.5.10 [*Preheat and Interpass Temperatures*] of this Schedule.

4-4.14.5.9 Grinding of Welds

A. Welds that are sufficiently smooth with a neat appearance and uniform profile will not require grinding.

   1. Welds not conforming to an acceptable profile must be ground to the proper profile without damaging or substantial removal of the base metal.

B. Grinding must be smooth and parallel to the line of stress. Caution must be exercised to prevent over grinding.

   1. Over grinding that results in reduced thickness of the base metal or size of the weld is not permitted.

   2. Acceptability of the welds without grinding will be determined by the City, in its discretion.

4-4.14.5.10 Arc Strikes

A. Arc strikes are not permitted.

   1. In the event of accidental arc strikes a repair procedure must be prepared and authenticated by a Professional Engineer.

   2. The repair procedure must include the complete grinding out of the crater produced by the arc strike.

3. The repair procedure must include MPI and hardness testing of the affected area. Hardness of the repaired area must conform to the requirements of Section 4-4.11.9.6 [*Hardness Tests*] of this Schedule.

4-4.14.5.11 Methods of Weld Repair

A. Repair procedures for damaged base metal and unsatisfactory welds must be prepared and authenticated by a Professional Engineer prior to repair work commencing.

4-4.14.5.12 Plug and Slot Welds

A. Plug welds or slot welds are not permitted.

4-4.14.6 Fabrication

4-4.14.6.1 General

A. The fabrication of bearings must be carried out as required to achieve the required performance of the bearings.

B. Fabrication of plain and steel reinforced elastomeric bearings and pot bearings must comply with the following:

1. AASHTO LRFD Bridge Construction Specifications (AASHTO LRFD BCS)

2. AASHTO M251 – Standard Specifications for Transportation Materials and Methods of Sampling and Testing – Standard Specification for Plain and Laminated Elastomeric Bridge Bearings

3. AWS – Bridge Welding Code D1.5

C. The fabricator for the steel bearing components must be approved by the CWB in accordance with CAN/CSA W47.1 in Divisions One or Two.

D. Fabrication must be performed in a fully enclosed area which is heated to at least 10ºC.

E. Only welders, welding operators and tackers approved by the CWB in the particular weld category to be carried out must be permitted to perform weldments.

4-4.14.6.2 Plain Bearings

A. Plain bearing pads must be moulded individually, cut from moulded strips or slabs of the required thickness, or extruded and cut to length.

4-4.14.6.3 Steel Laminated Bearings

A. Steel laminated bearings must be moulded under pressure as a single unit and heated in moulds that have a smooth surface finish.

B. The steel laminates must be a uniform 3 mm nominal thickness without any sharp edges.

C. The bond between the elastomer and the steel laminates must be such that when a sample is tested for separation, failure must occur within the elastomer and not between the elastomer and steel laminate.

D. The 2.5 mm deep recess in the top 9.5 mm steel laminate for sliding bearings must be machined in accordance with Section 4-4.14.6.5 [*Machining*] of this Schedule.

4-4.14.6.4 Pot Bearings

A. Stainless steel sheets in contact with PTFE must be continuously welded around the perimeter to the backing plate to prevent ingress of moisture.

   1. The weld must be clean, uniform, and without overlaps and must be located outside the area in contact with PTFE.

B. The threaded portion of the bolts must be coated with silicone grease prior to installation.

C. Virgin or glass filled PTFE elements must be recessed in a rigid backing material and must be bonded over the entire area with an adhesive.

   1. The rigid backing material must be grit blasted and cleaned with oil free compressed air prior to applying the adhesive.

D. The PTFE elements used as mating surfaces for guides for lateral restraint must extend to within 10 mm from the ends of the backing plates.

4-4.14.6.5 Machining

A. All metal to metal contact surfaces must be machined.

   1. Machining must be done after welding.

   2. For pot bearings, the pots and pistons must be machined from solid metal plate or castings.

B. There must be no openings or discontinuities in the metal surfaces in contact with the confined elastomer or PTFE.

C. The surface finish of metal plate in contact with any metal plate or confined elastomer in pot bearings and sliding laminated elastomeric bearings must be machined to a surface finish of 6.4 µm and a flatness tolerance of 0.001 x bearing dimension.

4-4.14.6.6 Identification

A. Each bearing must be marked with the fabricator's name, date of manufacture and unique identification number.

   1. The characters must not be less than 10 mm in height.

4-4.14.6.7 Galvanizing

A. The fabricator must provide a smooth finish on all edges and surfaces, and remove all weld spatters, and all welding flux residue from the steel components prior to galvanizing.

B. Unless otherwise specified, metal components, except weathering steel (CAN/CSA G40.20/G40.21 350A and 350AT) and stainless steel, must be hot-dip galvanized after fabrication in accordance with ASTM A123/A123M Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products and ASTM F2329 Standard Specification for Zinc Coating Hot-Dip Requirements for Application to Carbon and Alloy Steel Bolts, Screws, Washers, Nuts, and Special Threaded Fasteners or metallized in accordance with ASTM A780, Method A3.

C. The cleaning and pickling procedure of high strength ASTM A193 Grade B7 anchor rods must be modified prior to hot-dip galvanizing:

   1. brush blast to remove mill scale and oil after threading ends;

2. flash pickle up to five minutes; and

3. quick dry prior to hot-dip galvanizing (not stored in flux or acid rinse).

D. For pot bearings, the pot and piston plates, except surfaces in contact with elastomer, must be metallized in accordance with ASTM A780, Method A3 with the thickness of metallizing not less than 180 µm.

E. Galvanizing repairs must provide a coating that has a minimum thickness of 180 µm, adheres to the member and has a finished appearance similar to that of the adjacent galvanizing.

1. Galvanizing repair must comply with ASTM A780, Method A3 “Metallizing” unless the area requiring repair does not exceed 100 mm2 in which case the repairs may comply with ASTM A780 Method A1 “Repair Using Zine-Based Alloy”.

2. Galvanizing repairs must be tested for adhesion.

3. Repairs may require complete removal of the galvanized coating and re-galvanizing.

F. The galvanized contact surfaces of bolted connections must be hand wire brushed to a Class A slip coefficient surface condition.

1. Slip coefficient surface conditions must meet the requirements of CAN/CSA S6 Table 10.9.

G. Galvanized material must be stacked or bundled and stored to prevent wet storage stain in accordance with the American Hot-Dip Galvanizers Association (AHDGA) publication “Wet Storage Stain”.

1. Any evidence of wet storage stain must be removed to the satisfaction of the City.

4-4.14.7 Base Plate Corrosion Protection

A. The bottom surface of each base plate must be protected by a medium grey colour barrier coating to prevent contact between the zinc and the concrete.

1. The galvanized surface must be roughened prior to application of barrier coating.

2. The surface preparation of the galvanized surface and the dry film thickness of the coating must be in accordance with the coating Manufacturer’s recommendations.

B. The fully cured coating must be tested for adhesion in accordance with ASTM D3359 “Standard Test Methods for Measuring Adhesion by Tape Test”.

1. The selected method of testing must depend on the dry film thickness of the coating.

2. The coating manufacturer’s product data sheets must be provided to the City prior to the application of the coating.

3. Adhesion test result must meet a minimum of “4B” classification (maximum allowable flaking of 5%).

4-4.14.8 Tolerances

A. Plain and steel laminated bearing tolerances must comply with AASHTO M251.

B. Pot bearing tolerances must be as follows:

1. the deviation from flatness of PTFE surfaces must not exceed:

a. 0.2 mm, when the diameter or diagonal is equal to or less than 800 mm; or

b. 0.00025 of the diameter or diagonal, when the diameter or diagonal is greater than 800 mm.

2. The deviation from flatness of stainless steel surfaces in contact with PTFE for plane surfaces and from the theoretical surface for spherical surfaces must not exceed:

   a. 0.0003 LH mm for a rectangular PTFE element; or

   b. 0.0006 RH mm for a circular PTFE element.

   c. where:

      i. L = the greater plan dimension for a rectangular bearing

      ii. R = the radius of a circular bearing

      iii. H = the free height of PTFE element

3. For pot bearings, the tolerance of fit between the piston and the pot must be + 0.50 to + 1.25 mm. The inside diameter of the pot cylinder must be the same as the nominal diameter of the elastomer and must be machined to a tolerance of:

   a. 0 to + 0.125 mm for diameters up to and including 500 mm; or

   b. 0 to + 0.175 mm for diameters over 500 mm.

4. The plan dimensions of the recess for PTFE must be the same as the nominal plan dimensions of the PTFE and must be machined to a tolerance of 0 to + 0.2% of the diameter or diagonal.

   a. overall bearing plan dimension ± 3 mm

   b. overall bearing height ± 3 mm

   c. machined surface dimensions ± 0.4 mm

5. Elastomeric components must meet the following requirements:

   a. diameter:

      i. 0.0 to - 1.5 mm for diameters ≤ 500 mm; or

      ii. 0.0 to - 2.0 mm for diameters > 500 mm

   b. thickness 0.0 to + 1.0 mm.

6. Brass rings must meet the following requirements.

   a. Difference between internal diameter of brass ring and diameter of recess in the moulded elastomer must be 0 to + 0.5 mm.

   b. Difference between sum of thicknesses of brass rings and recess depth in the moulded elastomer 0 to + 0.25 mm.

7. Recessed guide bars must meet the requirements of the American Standard Clearance Locational Fit Class LC3 according to ANSI B4.1.

8. Guides for lateral restraints must have a 0.50 mm ± 0.25 mm gap between metal restraints surfaces and mating PTFE elements.

9. PTFE components must meet the following requirements:

    a. the plan dimension of the PTFE must be 0 to – 0.2% of the design diameter or diagonal;

    b. the thickness of the PTFE must be within 0 to + 10.0% of the design thickness; and

    c. the depth of recess of the PTFE must be 0 to + 0.3 mm.

4-4.14.9 Inspection and Testing

A. Inspection and testing must be carried out as required to ensure that the bearings have the required properties.

B. An independent accredited testing company must be engaged to perform the testing of the bearing materials and the finished bearings.

    1. The inspection and testing results and the manufacturer’s certification, as a written affidavit that the material supplied meets the Project Requirements, must be provided to the City.

C. 25% of all fillet and partial penetration welds must be magnetic particle inspected in accordance with ASTM E-709.

D. Elastomeric Bearings

    1. Testing of elastomeric compounds must be completed in accordance with Section 8 of AASHTO M251. Material must conform to the specified requirements of Table X1.

    2. Testing of the completed bearings must be in accordance with AASHTO M251 with the exception that contrary to Sections 8.8.1 and 8.8.2 of AASHTO M251, testing of all bearings is required.

        a. The optional testing described in Section 8.9 of AASHTO M251 is not required.

        b. The dimensional tolerances for each bearing must be checked.

        c. The hardness of elastomer must be tested.

        d. A minimum of two sample laminated bearings must be cut and tested for shear modulus.

    3. The increment in compressive deformation of laminated bearings must not exceed 0.05 of the effective rubber thickness, when the bearing load is increased from an initial pressure of 1.5 MPa to a pressure of 7 MPa when tested in accordance with the requirements of Section 9.1 of AASHTO M251.

E. Pot Bearings

    1. Testing of elastomeric compounds must be completed in accordance with AASHTO M251.

    2. Testing of the finished bearings must be completed in accordance with the requirements of Section 18.3.4 of the AASHTO LRFD BCS.

    3. The long-term deterioration test described in Section 18.3.4.4.3 of the AASHTO LRFD BCS is not required.

    4. The proof load test described in Section 18.3.4.4.4 of the AASHTO LRFD BCS must be carried out in accordance with the long-term proof load test requirements.

4-4.14.9.1 Fabrication Outside of Canada

A. All components fabricated outside of Canada must be shipped to a shop located in Canada that is certified by CWB in accordance with CSA W47.1 to Division One or Two for re-inspection and testing.

B. The components must be in a condition that facilitates all re-inspection and testing requirements.

C. The re-inspection and testing at the Canadian shop must be completed in accordance with Section 4-4.11.9 [*Inspection and Testing*] of this Schedule.

D. The component must also be inspected by a CSA 178.2 Level III certified welding inspector accredited with W47.1 to inspect the following items:

1. ensure that all components are clean and undamaged following transportation; and

2. 100% of all fillet and partial penetration welds using magnetic particle inspection in accordance with ASTM E709.

E. Components must not be shipped from the Canadian shop until all requirements have been met and mill certificates have been provided to the City.

4-4.14.10 Installation

4-4.14.10.1 General

A. Bearings must be installed in a manner that does not damage them or affect their performance.

B. Bearings must be adjustable in accordance with the requirements in Section 4-3.16 [*Bearings*] of this Schedule.

C. A bearing installation procedure must be prepared prior to installation.

4-4.14.10.2 Bearing Installation Procedure

A. The Design-Builder must submit a bearing installation procedure for laminated elastomeric bearings and pot bearings to the Designer for review and acceptance prior to the commencement of bearing installation work. The installation procedure must include drawings and documents necessary to describe the following:

1. details of bearing storage and protection;

2. survey information for location and elevation of shim stacks, grout pads and anchor rod voids;

3. temporary support and stability measures (hold point);

4. bearing elevation, location, grade and orientation verification (hold point);

5. setting expansion bearings for temperature (witness point);

6. schedule of anchor rod grouting;

7. methods of forming, placing, curing, and sealing of grout pads; and

8. enclosure and system of heating for grouting in cold weather.

B. The bearing installation procedure will be considered a Professional Work Product and must be authenticated by a Professional Engineer licensed to practice in the Province of Alberta and validated by a Responsible Member, in accordance with APEGA requirements.

4-4.14.10.3 Bearing and Anchorage

A. Bearing base plates must not be placed upon surfaces which are improperly finished, deformed or irregular.

B. Foreign materials on concrete surfaces, such as oils, grease or other contaminants must be removed by sandblasting prior to installation of anchor rods.

C. Field welding adjacent to elastomeric pads must be performed so as not to damage the elastomer.

   1. The temperature of the steel adjacent to the elastomer must be kept below 120°C.

   2. The distance between the weld and the elastomer must be at least 40 mm.

D. The tops of bearing sole plates must be within a tolerance of ±3 mm of the correct elevation prior to girder erection.

E. The attachment of sole plates to girders by welding must be in the longitudinal direction along the edge of the bottom flange or shoe plate.

   1. Transverse welding is not permitted.

   2. Transverse ends must be sealed with Sikaflex 1a or equivalent caulking material.

F. Galvanizing or metallizing damaged during bearing installation must be repaired in accordance with the requirements of ASTM A780/A780M, Method A3.

4-4.14.10.4 Grout Pads

A. Grout pads must be constructed using a flowable non-shrink grout from the Alberta Transportation Products List.

   1. Dry-pack methods of constructing grout pads are not permitted.

   2. Filling of anchor rod voids and construction of grout pads must be done by workers competent in this work.

B. Grout must be packaged in waterproof containers with the production date and shelf life of the material shown.

C. Grout must be mixed, placed, and cured in accordance with the manufacturer's recommendations stated on their published product data sheet. Curing must be a minimum 3-day wet cure.

D. A set of compressive strength cubes must be taken to represent each day’s grout production or each 0.25 m^3 of grout placed, whichever is more frequent.

E. Prior to casting deck concrete, the average minimum compressive strength of three grout cubes at 28 days must be a minimum of 30 MPa measured in accordance with CAN/CSA A23.2-1B.

4-4.14.10.5 Grouting in Cold Weather

A. When the daily minimum air temperature or the temperature of the girders, bearings or substructure concrete in the immediate area of the grouting is, or is expected to be below 5°C during the placing and curing period, the following provisions for cold weather grouting must be applied:

   1. Before grouting, adequate preheat must be provided to raise the temperature of the adjacent areas of the girders, bearings and substructure concrete to at least 15°C.

2. The temperature of the grout during placing must be between 10°C and 25°C.
3. The grout pads must be enclosed and kept at 15°C to 25°C for a minimum of 5 days.
4. The enclosure must meet the requirements of Section 4-4.6.18 [*Concreting in Cold Weather*] of this Schedule.

4-4.15 MECHANICALLY STABILIZED EARTH WALLS

4-4.15.1 General

A. This Section 4-4.15 [*Mechanically Stabilized Earth Walls*] sets out the requirements for all MSE retaining walls forming part of a Structure, including minimum requirements for the supply, fabrication and construction of the walls.

4-4.15.2 Engineering Data

4-4.15.2.1 Shop Drawings

A. Shop drawings must be prepared for all MSE walls prior to fabrication and submitted to the City. As a minimum, the shop drawings must show the following:
   1. MSE wall design criteria and material lists;
   2. precast concrete fascia panel reinforcing, connection and hardware details;
   3. MSE wall layout plans and elevations complete with dimensions, elevations and typical wall cross-sections;
   4. MSE wall backfill properties;
   5. all MSE wall component and connection details; and
   6. drainage and site drainage details.

B. Shop drawings must be authenticated by a Professional Engineer and must be submitted to the City.

4-4.15.3 Materials

4-4.15.3.1 Concrete

A. Concrete must comply with Section 4-4.6 [*Cast-In-Place Concrete*] and Section 4-4.7 [*Precast Concrete*] of this Schedule as applicable.

4-4.15.3.2 Concrete Reinforcement

A. Concrete reinforcement must comply with Section 4-4.10 [*Concrete Reinforcement*] of this Schedule.

4-4.15.3.3 Soil Reinforcing

A. Steel soil reinforcing, including inspection wires, must comply with ASTM A1064/A1064M.

B. Steel soil reinforcement consisting of steel strip reinforcement must meet the requirements of ASTM A572/A572M.

C. Safeguarding measures to prevent embrittlement and testing to detect embrittlement must be completed in accordance with ASTM A143/A143M for all lots of steel reinforcement, connections and associated hardware.

D. Galvanizing of steel soil reinforcing must comply with ASTM A123/A123M, and ASTM F2329/F2329M.

1. All damage to galvanizing must be repaired in accordance with ASTM A780/A780M, Method A3 "Metallizing" unless the repair area is less than 100 mm^2 in which case the repairs may comply with ASTM A780 Method A1 "Repair Using Zine-Based Alloy". The thickness of the coating of both methods must be 180 μm, and the repair tested for adhesion.

E. Galvanized material must be stacked or bundled and stored to prevent wet storage stain in accordance with the AHDGA publication "Wet Storage Stain".

F. Geosynthetic soil reinforcing must meet the requirements of AASHTO LRFD, Section 11.10.6.4.3b.

1. The requirements "for applications involving severe consequences of poor performance or failure" must be applied.

2. Site specific studies and testing must be carried out to determine the strength reduction factors for geosynthetic reinforcements used in MSE wall construction to account for short-term and long-term degradation due to installation damage (RF_{ID}), creep (RF_{CR}), and chemical and biological factors (RF_D) throughout the Design Life. The studies and testing must be performed by a third party, independent agency accredited by the Standard Council of Canada.

G. Geosynthetic soil reinforcing materials must meet the requirements of the following tests:

1. ASTM D7737-11 – Standard Test Method for Individual Geogrid Junction Strength

2. ASTM D5262 – 07(2012) – Standard Test Method for Evaluating the Unconfined Tension Creep and Creep Rupture Behavior of Geosynthetics

3. ASTM D6706 – 01(2013) – Standard Test Method for Measuring Geosynthetic Pullout Resistance in Soil

4. GG 2-87 – Standard Test Method for Geogrid Rib Junction Strength

5. GG4-05 – Standard Practice for Determination of the Long-Term Creep Design Strengths of Geogrids

6. GG4(a) Revised 2012 – Standard Practice for Determination of the Long-Term Design Strength of Stiff Geogrids

7. GG4(b) Revised 2012 – Standard Practice for Determination of the Long-Term Design Strength of Flexible Geogrids

H. Geosynthetic soil reinforcing materials must contain stabilizers or inhibitors to prevent degradation of properties due to ultraviolet light exposure.

I. The ultimate tensile strength (T) of the specific soil reinforcing products used in MSE wall construction must be determined by an independent agency such as the HITEC or the AASHTO National Transportation Product Evaluation Program.

1. Product lines must have been tested within the last 3 years.

J. Material test reports must be submitted for all geosynthetic soil reinforcement and impermeable geomembrane.

K. Mill test reports must be submitted for all steel fabricated components, steel soil reinforcement connections, and associated hardware and as a minimum, must include the following items:

1. heat number;
2. date;
3. location of production;
4. compliance with production standards;
5. chemical analysis;
6. mechanical properties; and
7. galvanizing processes.

L. Mill test reports originating from a mill outside of Canada or the United States of America must meet the requirements of Section 4-4.11.3.4 [*Mill Certificates*] of this Schedule.

4-4.15.3.4 Backfill

A. MSE wall reinforced backfill must be "Crushed Aggregate Material" complying with Table 4-4.15.3-1 [*Class Designation of MSE Wall Backfill Materials*] and must be free of organic matter and other deleterious substances.

Table 4-4.15.3-1 Class Designation of MSE Wall Backfill Materials

Metric Sieve Size (CGSB 8-GP-2M)	Crushed Aggregate Material Des 2 Class 20	Crushed Aggregate Material Des 2 Class 25	Crushed Aggregate Material Des 2 Class 40
Sieve Size µm	**Percent Passing**	**Percent Passing**	**Percent Passing**
40 000			100
25 000	100	100	70 - 94
20 000	100	82 - 97	N/A
16 000	84 - 94	70 - 94	55 - 85
10 000	63 - 86	52 - 79	44 - 74
5 000	40 - 67	35 - 64	32 - 62
1 250	22 - 43	18 - 43	17 - 43
630	14 - 34	12 - 34	12 - 34
315	9 - 26	8 - 26	8 - 26
160	5 - 18	5 - 18	5 -18
80	2 - 10	2 - 10	2 - 10
% fractures by weight (2 faces)	60+	60+	50+
Plasticity Index	NP - 6	NP - 6	NP - 6
L.A. Abrasion Loss Percent Maximum	50	50	50

B. Laboratory density testing must be completed on backfill source(s) in accordance with ASTM D698.

C. MSE wall backfill material placed within 2.0 m of the MSE wall face must be free draining and have no more than 5% passing the 80 μm sieve size.

1. Soil filters between soil zones must be designed to prevent infiltration migration of fine soil particles between the zones.

D. MSE wall backfill material containing steel soil reinforcing must comply with Table 4-4.15.3-2 [*Electrochemical Parameters for MSE Wall Steel Soil Reinforcing*].

Table 4-4.15.3-2 Electrochemical Parameters for MSE Wall Steel Soil Reinforcing

Select Backfill Requirements		Test Method (ASTM)	Test Method (AASHTO)
Resistivity	≥ 3000 ohm-cm	G57	T 288
pH	5 - 10	G51	T 289
Chlorides	≤ 100 ppm	D512	T 291
Magnesium Sulphate Soundness	Loss less than 30% after four cycles	D5262	T 104
Sulphates	≤ 200 ppm	D516	T 290
Organic Content	≤ 1.0%	D2974	T 267

E. MSE wall backfill material containing geosynthetic soil reinforcing must comply with Table 4-4.15.3-3 [*Requirements for Geosynthetic Reinforcing*].

Table 4-4.15.3-3 Requirements for Geosynthetic Reinforcing

Select Backfill Requirements		Test Method (ASTM)	Test Method (AASHTO)
pH	3 - 12	G51	T 289
Organic Content	≤ 1.0%	D2974	N/A
Design Temperature at the Wall Site	≤ 30°C	N/A	N/A

F. The collection of backfill samples for testing must be from the stockpiles at the top, middle and bottom portions and approximately 0.6 m from the face of the stockpile.

1. Resistivity Testing must be carried out on six samples (two top, two middle, two bottom).

2. pH, chloride, sulphate, and organic content testing must be carried out on nine samples (three top, three middle, three bottom).

4-4.15.3.5 Geotextile Filter Fabric

A. Non-woven geotextile filter fabric must comply with Table 4-4.15.3-4 [*Specification for Non-Woven Geotextile Filter Fabric*].

Table 4-4.15.3-4 Specification for Non-Woven Geotextile Filter Fabric

Non-Woven Geotextile Filter Fabric		
Specifications and Physical Properties		**Test Method (ASTM)**
Grab Strength Elongation (Failure) CBR Puncture Strength Trapezoidal Tear	≥ 650 N ≥ 50% ≥ 250 N ≥ 275 N	D4632 D4632 D6241 D4533
Minimum Fabric Lap to be 300 mm		

4-4.15.3.6 Impermeable Geomembrane

A. Impermeable geomembrane must be PVC, HDPE or LLDPE geomembrane with a minimum thickness of 0.75 mm and must comply with Table 4-4.15.3-5 [*Specification for Impermeable Geomembrane*].

Table 4-4.15.3-5 Specification for Impermeable Geomembrane

Impervious Geomembrane		
Specifications and Physical Properties		**Test Method (ASTM)**
Tear Strength CBR Puncture Strength	≥ 45 N ≥ 140N	D1004 D4833

1. All seams in the membrane must be welded or bonded in accordance with the manufacturer's recommendations to prevent leakage.

4-4.15.4 Type 1c Concrete Sealer

A. Type 1c sealer must be applied to exposed concrete surfaces.

4-4.15.5 Storage and Handling

A. All materials must be protected from damage during storage and handling.

1. All materials must be stored above ground and covered and protected from rain, snow, dirt and ultraviolet light.
2. Precast concrete fascia panels must be stored such that the uniform colour of the panels is maintained and protected from staining or discoloration.

4-4.15.6 MSE Wall Panel Production

A. The fabrication of precast concrete MSE wall panels must comply with Section 4-4.7 [*Precast Concrete*] of this Schedule.

4-4.15.7 Inspection and Testing

A. Backfill compaction testing of the reinforced backfill must be carried out at a minimum frequency of one test per lift for every 45 m of wall length or part thereof with no less than one test per day.

B. The backfill must be tested in accordance with the requirements of Table 4-4.15.7-1 [*Sampling and Testing of Backfill Properties During Construction*].

Table 4-4.15.7-1 Sampling and Testing of Backfill Properties During Construction

Range of Resistivity (ohm-cm)	Sample Interval for Resistivity Testing (m3)	Sample Interval for PH, Chlorides, Sulphates, Organic Testing (m3)
> 5000	3000	1500
< 5000 and ≥ 3000	1500	750

4-4.15.8 Construction

4-4.15.8.1 General

A. The construction of the MSE wall system must conform to the details on the shop drawings and be in compliance with the supplier's recommendations.

1. The supplier of the MSE wall system must provide a full-time qualified representative on-site during construction to advise regarding construction procedures and to monitor that the MSE wall construction is being carried out in accordance with the shop drawings and supplier's recommendations.

2. The representative of the MSE wall supplier must document any deviations from shop drawings, recommended construction procedures and accepted industry practice, and must record the mitigation measures implemented to correct such deviations.

3. The MSE wall supplier representative must sign off the final construction details.

4-4.15.8.2 Levelling Pads

A. The foundation subgrade must be proof rolled to identify any soft spots. Soft material must be removed and replaced with compacted granular material to the satisfaction of the Designer.

B. The concrete levelling pads must be placed to the grades and lines shown on the applicable final Design Data.

1. When checked with a 3 m long straight edge there must not be a gap greater than 3 mm between the top of the levelling pad and the straight edge.

C. Concrete levelling pads must project at least 75 mm past each side of the precast concrete MSE wall panels.

D. After the erection of the first row of MSE wall panels, any openings between the levelling pad steps must be filled.

4-4.15.8.3 Backfill

A. Backfill must be placed in conformance with Section 4-4.1 [*Backfill*] of this Schedule and the MSE wall supplier's specifications.

1. Backfill placement must closely follow erection of each course of MSE wall panels.

B. Backfill must be placed in such a manner as to avoid any damage, disturbances or misalignment of the MSE wall face panels and such that soil reinforcement is fully supported over its length.

   1. Any MSE wall components that are damaged must be removed and replaced.

   2. Any misalignment or distortion of the precast concrete MSE wall panels must be corrected before continuing with the Project Work.

C. Backfill must be compacted in lifts not exceeding 150 mm in thickness of loose material.

D. Backfill must not be placed on frozen substrate.

E. Overlapping geosynthetic reinforcement layers must be separated by a minimum 75 mm of compacted backfill.

F. A control strip density must be established on the first backfill lift and every 900 mm (vertically) of backfill placed thereafter for every 45 m of wall or part thereof and not less than once per day. The control strip density must be re-established where the gradation or source of aggregate change, or when different compaction equipment is used.

   1. The control strip density must be measured in accordance with Alberta Transportation Test Method ATT-58A, Density Test Control Strip Method.

   2. All backfill lifts must be compacted to a minimum of 98% of the control strip density and must be measured in accordance with Alberta Transportation Test Method ATT-11, Density Test In-Place Nuclear Method.

G. Backfill compaction must be performed in such a manner that the equipment moves in a direction parallel to the MSE wall panels or away from the MSE wall panels toward the end of the soil reinforcing.

   1. Equipment must not be allowed to run directly on the soil reinforcing.

   2. Only hand operated power tampers and vibrators must be used for compaction within 1000 mm of the MSE wall panels.

   3. At the completion of each day’s work, the backfill material must be sloped away from the MSE wall panels to direct potential run-off away from the wall face.

   4. Surface run-off from adjacent areas into the MSE wall construction site is not permitted.

H. Sieve analysis must be completed on backfill being placed at the beginning and end of each day for each zone of backfill containing soil reinforcing.

4-4.15.8.4 Precast Concrete MSE Wall Panel Placement Tolerances

A. Precast concrete MSE wall panel placement tolerances after installation must be:

   1. the out-of-flatness of wall surfaces measured in any direction must not exceed 25 mm under a 3 m straight edge;

   2. the offset of adjacent panel edges at joints must not exceed 10 mm;

   3. the overall vertical alignment of the completed wall must not be out-of-vertical by more than 4 mm/m of wall height; and

   4. the joint width between MSE wall panels must be between 10 mm and 30 mm.

4-4.15.8.5 Impermeable Geomembrane

A. Where required, impermeable membrane must be installed so as to prevent leakage through the membrane and to direct drainage away from the MSE wall panels and soil reinforcing.

B. Seams of impermeable geomembranes must be placed parallel to the MSE wall and lapped in the direction of Positive Drainage to produce a shingling effect.

4-4.16 DECK WATERPROOFING MEMBRANE SYSTEM

4-4.16.1 General

A. This Section 4-4.16 [*Deck Waterproofing Membrane System*] sets out the requirements for all deck waterproofing membrane and ACP forming part of the 111 Street Roadway Bridge Widening, including minimum requirements for quality, supply, handing and placing of deck waterproofing membrane and ACP.

1. This includes the supply and installation of the deck waterproofing membrane and ACP system shown on *Alberta Transportation Standard Drawings* S-1838, S-1839 and S-1440 (Standard Water Proofing System for Deck and Abutments).

4-4.16.2 Engineering Data

A. Documentation showing that the deck waterproofing membrane materials comply with the requirements of Section 4-4.16.3 [*Materials*] of this Schedule must be obtained prior to the installation of the deck waterproofing membrane.

B. An asphalt mix design for the 111 Street Roadway Bridge Widening, with applicable material quality compliance test reports for each type of ACP, must be submitted before first placement of such ACP.

4-4.16.3 Materials

4-4.16.3.1 General

A. Materials supplied must be able to withstand the heat generated during the deck waterproofing membrane and ACP installation processes without affecting the performance of the material.

4-4.16.3.2 Deck Waterproofing Membrane Tack Coat

A. The tack coat must be a primer type meeting the requirements of CAN/CGSB-37-GP-9MA.

4-4.16.3.3 Asphalt Membrane

A. Asphalt membrane must be hot applied rubberized asphalt complying with the Ontario Ministry of Transportation's OPSS 1213 Specification.

1. Asphalt membrane materials must be supplied in cakes that are sealed and labelled by the manufacturer.

4-4.16.3.4 Rubber Membrane

A. Rubber membrane must consist of 1.2 mm thick butyl and ethylene propylene diene monomer rubber.

1. The rubber membrane must comply with CAN/CGSB 37.52M.

4-4.16.3.5 Membrane Reinforcing Fabric

A. Membrane reinforcing fabric must consist of spun bonded sheet structure composed of 100% continuous filament polyester fibres bonded together at their crossover points.

1. Membrane reinforcing fabric must be supplied in minimum widths of 300 mm.

4-4.16.3.6 Wick Drain

A. Wick drain must consist of composite polypropylene with a total thickness of 3.6 mm and supplied in 100 mm widths.

1. The wick drain puncture strength must be a minimum of 45 N measured in accordance with ASTM D4833.

4-4.16.3.7 Waterproofing Protection Board

A. Waterproofing protection board must comply with the Ontario Ministry of Transportation's OPSS 1215 Specification for Protection Board.

1. Waterproofing protection board must consist of panels that provide a protective cushion between the hot mix ACP and the asphalt membrane.

4-4.16.3.8 ACP

1. ACP materials must comply with Section 4-4.13 [*Asphalt Concrete Pavement (ACP)*] of this Schedule unless otherwise specified in this Section 4-4.16 [*Deck Waterproofing Membrane System*] of this Schedule.

4-4.16.4 Equipment

4-4.16.4.1 General

A. Equipment and methods used to place the deck waterproofing membrane and ACP must be adequate to produce and place the materials as specified in this Section 4-4.16 [*Deck Waterproofing Membrane System*] of this Schedule.

4-4.16.4.2 Heating and Mixing Kettle

A. A heating and mixing kettle must be used to heat the asphalt membrane.

1. The kettle must be capable of keeping the contents continuously agitated, free flowing and lump free until the material is drawn for application.
2. The kettle must be a double boiler oil transfer type with a built-in agitator and must be equipped with permanently installed dial type thermometers with an accuracy of ± 2°C to measure the temperature of the melted compound and oil.
3. A separate calibrated thermometer with an accuracy of ± 2°C must be available on -site to verify material temperatures.

4-4.16.5 Inspection and Testing

4-4.16.5.1 Deck Waterproofing Membrane

A. The asphalt membrane, rubber membrane, membrane reinforcing fabric and protection board must be tested to verify compliance with Ontario Provincial Standard Specifications OPSS 1213 and OPSS 1215.

4-4.16.5.2 ACP

A. Inspection and testing of ACP must comply with the D&CS unless otherwise specified in this Section 4-4.16 [*Deck Waterproofing Membrane System*] of this Schedule.

B. The ACP must meet the quality control requirements of the D&CS unless otherwise specified in this Section 4-4.16 [*Deck Waterproofing Membrane System*] of this Schedule.

4-4.16.5.3 Installation of Deck Waterproofing Membrane

A. Installation of the deck waterproofing membrane must only be carried out when the air and concrete surface temperatures are 5°C or higher.

B. The operations involved in installing the deck waterproofing membrane must be performed in sequential order, such that there are no delays between individual operations except those necessary to meet the requirements of this Section 4-4.16 [*Deck Waterproofing Membrane System*] of this Schedule.

C. All traffic, other than the construction equipment directly associated with the installation of the deck waterproofing membrane and ACP must be restricted from traveling over the prepared deck waterproofing membrane areas.

    1. These restrictions must remain in place until after the final lift of ACP has been placed over the deck waterproofing membrane and cooled to ambient temperature.

4-4.16.5.4 Surface Preparation

A. Concrete surfaces receiving a deck waterproofing membrane must be cured at least 14 days and then allowed to dry for a minimum of 3 days before commencing installation of the deck waterproofing membrane.

    1. Drying of the concrete deck surface by use of torches or other means that might be harmful to the deck is not permitted.

    2. Installation of the deck waterproofing membrane, including tack coating must not commence until the concrete surface is fully dry and clean.

B. Once the surface of the concrete is completely dry it must be sandblasted or shotblasted as required to expose sound, laitance free concrete over the entire installation area.

    1. All dirt and debris on the concrete surface must be removed and disposed of leaving a prepared surface satisfactory for tack coating.

4-4.16.5.5 Tack Coating for Deck Waterproofing Membrane

A. Tack coat must be applied after the City has accepted the surface preparation work.

B. Tack coat must be applied to the concrete surface wherever deck waterproofing membrane is required.

C. All concrete surfaces must have less than 6% moisture prior to application of the tack coat.

    1. Testing must be completed using a Hygrometer or Protimeter.

D. Immediately prior to the application of the tack coat, the concrete surface must be blown clean with oil and water free compressed air to remove all dust and any other foreign material.

E. The tack coat must be cut back with an equal volume of gasoline type solvent or alternative cut back asphalt product that is compatible with the asphalt membrane.

1. The tack coat application rate must be such that the tack material will be absorbed into the concrete, resulting in a surface that is dull and black in appearance.

2. Excess application of tack coat, as indicated by a shiny black surface, is not permitted.

3. Tack coat material must be applied at an approximate rate of 0.25 L/m2.

F. Waterproofing equipment or material is not permitted on the tack coat until it has fully cured and is completely tack free.

4-4.16.5.6 Waterproofing of Joints and Cracks

A. After tack coat application and prior to application of the primary asphalt membrane to the deck, a coat of asphalt membrane 3 to 4 mm thick must be applied over each joint and crack including over construction joints, lifting hook pockets and concrete patch joints. The membrane must be wide enough to extend 200 mm on either side of each joint or crack and must be applied in accordance with Section 4-4.16.5.8 [*Application of Asphalt Membrane*] of this Schedule.

B. Membrane reinforcing fabric must be placed in the asphalt membrane over the joints and cracks.

1. The strips of membrane reinforcing fabric material must be wide enough to extend 150 mm on either side of the joints and cracks and must be applied while the asphalt membrane is still hot and tacky.

2. Membrane reinforcing fabric strips must be overlapped a minimum of 100 mm where multiple strips are used.

3. The membrane reinforcing fabric must be covered with an additional layer of asphalt membrane 2 to 3 mm thick.

C. Along curbs, barriers, medians, deck drains and deck joints asphalt membrane 3 to 4 mm thick must be applied to the height of the top of the ACP surface course, and 150 mm onto the deck.

1. Rubber membrane must be applied into the first coat of asphalt membrane while it is still hot and sticky.

2. The rubber membrane must extend 50 mm up the vertical face of the curbs, barriers, medians, deck drains and deck joints, and 100 mm onto the deck surface.

3. Rubber membrane must be overlapped a minimum of 100 mm where multiple strips are used.

4. A second coat of asphalt membrane 2 to 3 mm thick must be applied to fully cover the rubber membrane.

4-4.16.5.7 Application of Asphalt Membrane

A. Asphalt membrane must not be applied until the tack coat has cured completely.

B. Cakes of asphalt membrane must be melted in the heating and mixing kettle to a temperature not exceeding that recommended by the membrane manufacturer.

1. The asphalt membrane must be applied within the temperature range recommended by the manufacturer.

C. The membrane must be applied in a uniform film having a minimum thickness of 4 mm and a maximum thickness of 6 mm.

D. The asphalt membrane must be applied in a continuous manner.

   1. Where joints in the asphalt membrane are unavoidable they must be overlapped by a minimum of 150 mm.

E. The asphalt membrane must be applied over all waterproofed joints and cracks, and must extend up the face of curbs, barriers, medians, deck drains and deck joints, to the height of the top of the ACP surface course.

F. Deck drains and drainage tubes must not be plugged by the asphalt membrane.

4-4.16.5.8 Installation of Wick Drain

A. Wick drains must be installed along the full length of gutters when the asphalt membrane is still hot and tacky.

   1. Special attention must be given to waterproofing and wick drain modifications required at deck drain locations.

4-4.16.5.9 Application of Protection Board

A. The asphalt membrane thickness must be checked and documented to confirm conformance to the requirements of Section 4-4.16.5.8 [*Application of Asphalt Membrane*] of this Schedule, prior to placing the protection board.

B. Protection boards must be laid on the asphalt membrane while the membrane is still hot, with the length of the board running transversely on the deck.

C. The protection boards must be placed with edges overlapping a minimum of 12 mm and a maximum of 25 mm both longitudinally and transversely. The protection board edges must be within 5 mm of all wick drains, faces of deck drains and faces at deck joints.

   1. Protection boards must be lapped to produce a shingling effect in both the longitudinal and transverse directions.

   2. Holes must be cut through the protection boards as required to allow water to drain freely through drainage tubes.

D. Protection boards must be placed such that the longitudinal (direction of traffic flow) joints are staggered at least 150 mm.

E. Boards must be rolled by means of a linoleum or lawn type roller while the asphalt membrane is still warm, in order to ensure good contact with the membrane.

   1. At locations where the edges of the protection board have curled up, the curled up edges must be cemented down using hot asphalt membrane material.

F. Protection boards that are warped, distorted or damaged in any way, whether by manufacture, storage, handling or exposure to weather, must be rejected.

4-4.17 SOIL NAILS

4-4.17.1 General

A. This Section 4-4.17 [*Soil Nails*] sets out the requirements for permanent soil nails and other related structural components resisting lateral earth load/surcharge load and forming part of a Structure, including minimum requirements for supply, installation, grouting and testing.

4-4.17.2 Engineering Data

4-4.17.2.1 Related Design-Build Construction Requirements

A. Section 4-4.6 [*Cast-In-Place Concrete*] of this Schedule.

B. Section 4-4.10 [*Concrete Reinforcement*] of this Schedule.

C. Section 4-4.18 [*Shotcrete*] of this Schedule.

4-4.17.2.2 Shop Drawings

A. Shop drawings showing fabrication and installation details of the soil nails must be submitted to the City. The shop drawings must include the following:

1. inclination, length, diameter, and vertical and horizontal spacing of soil nails;
2. type, length and size of the soil nail steel bar;
3. design pullout resistance per unit length of soil nail;
4. tendon anchorage details, including all components of soil nail head;
5. corrosion protection system for the tendons and soil nail head components;
6. the type and spacing of tendon centralizers and spacers; and
7. grout mix design and grout placement procedures.

4-4.17.2.3 Mill Certificates

A. Mill certificates for the soil nail tendons and couplers, including the ultimate strength, yield strength, load/elongation curves, and composition, must be provided to the City.

B. Mill certificates for the soil nail head components must be provided to the City.

C. Manufacturer certificates of compliance for the soil nail centralizers must be provided to the City.

D. Mill test reports originating from a mill outside of Canada or the United States of America must meet the requirements of Section 4-4.11.3.4 [*Mill Certificates*] of this Schedule.

4-4.17.3 Materials

A. Materials for soil nails must comply with Section 4-3.5.10 [*Soil Nails*] and Section 4-3.6.4.6 [*Soil Nail Corrosion Protection*] of this Schedule, and FHWA-NHI-14-007.

B. Tendon couplers must develop 120% of the specified tensile yield strength of the tendon as certified by the manufacturer.

C. Steel hardware of soil nail heads must be hot-dip galvanized per ASTM A153 / A153M.

D. Centralizers and spacers must be fabricated from Schedule 40 PVC pipe or tube, steel, or material non-detrimental to the steel bar. The use of wood is not permitted.

E. Soil nail grout must be neat cement or sand/cement mixture with a minimum 3-day compressive strength of 21 MPa and a minimum 28-day compressive strength of 30 MPa in accordance with ASTM C109.

F. Admixtures, if used, must meet the requirements of ASTM C494, and must be compatible with the grout and the steel bar components. The use of accelerators or expansive admixtures is not permitted.

4-4.17.4 Installation

4-4.17.4.1 General

A. The entity performing any soil nailing must be experienced in the construction and load testing of soil nails and have successfully constructed at least five projects in the last 5 years involving construction totaling at least 1000 soil nails of similar capacity to those required in the final Design Data.

B. Soil nail tendons, including all components of soil nail head, must be handled, stored and installed in such a manner as to avoid damage, corrosion or contamination with dirt or deleterious substances, in accordance with the requirements of FHWA-NHI-14-007.

C. The use of drilling fluids (such as bentonite slurry) to advance the soil nail holes is not permitted.

4-4.17.4.2 Installation Tolerances

A. Soil nail head location must not deviate from the design location by more than 150 mm in any direction.

B. Soil nail inclination must be within plus or minus three degrees from the design inclination.

C. Installation tolerances are applicable to individual soil nails, and not to the average of multiple soil nails in an area.

D. Soil nail tendon must be placed within 25 mm of the centre of the drill hole.

E. Soil nails that do not satisfy the specified tolerances must be replaced.

F. The soil nail head assembly must be installed perpendicular to the tendons, without bending or kinking of the tendons.

4-4.17.4.3 Grouting

A. Grouting of soil nails must comply with FHWA-NHI-14-007.

B. Grouting of drill holes after installation of the soil nail tendons must be completed within 2 hours of completion of drilling and must be done in one continuous operation. Cold joints in the grout column are not allowed except at the top of the bond length of production soil nails that will be proof tested.

C. The grout must be free of lumps and undispersed cement.

D. Admixtures, if used, must be mixed in accordance with the manufacturer's recommendations.

E. Soil nail grout must be tested in accordance with ASTM C109 at a frequency of no less than one test for every four m^3 of grout placed. Irrespective of the volume of grout placed, a minimum of one test must be performed on a set of grout cubes from each grout plant on each day of operation.

4-4.18 SHOTCRETE

4-4.18.1 General

A. This Section 4-4.18 [*Shotcrete*] sets out the requirements for shotcrete for soil nail walls including minimum requirements for quality, sampling and testing, placing, curing and finishing shotcrete.

4-4.18.2 Materials

4-4.18.2.1 Shotcrete

A. Only wet-mix shotcrete mix designs will be permitted.

B. A shotcrete mix design review letter, together with applicable material quality compliance test reports, must be submitted to the City.

C. The mix design review letter must include the following items:

1. an evaluation and summary of all mix constituents;
2. material test reports;
3. mix proportion quantities by mass or volumes;
4. cement type;
5. aggregate source, grading, and test reports;
6. water source(s); and
7. trial batch test results confirming the proposed mixture design is capable of meeting the specified performance requirements in Table 4-4.18.2.1 [*Concrete Classes*].

Table 4-4.18.2.1 Concrete Classes

Test Description	Test Method	Age (Days)	Specified Requirement
Maximum water/cementitious materials ratio			0.45
Air content – as shot	CSA A23.2-4C		4 +/- 1%
Slump at discharge into pump	CSA A23.2-5C		60 +/- 20 mm
Minimum compressive strength	CSA A23.2-14C	7 28	20 MPa 30 MPa
Maximum Boiled Absorption	CSA A23.2-11C	7	8%
Maximum Volume of Permeable Voids	CSA A23.2-11C	7	17%

4-4.18.2.2 Portland Cement

A. Portland cement must comply with CAN/CSA A3001. General use (Normal), Type GU, must be used unless otherwise specified herein.

1. Concrete intended for placement in sulphate environments may be produced with combinations of Type GU cement and supplementary cementing materials provided current CAN/CSA A3004-C8 test data demonstrates compliance with CAN/CSA A3001 requirements for high sulphate resistance.

4-4.18.2.3 Fly Ash

A. Fly ash, if required, must conform to the requirements of CSA A3001 Type F with a calcium oxide content not exceeding 12%.

B. A minimum Pozzolanic Activity Index of 75% at 28 days is required.

4-4.18.2.4 Silica Fume

A. Silica fume, if required, must conform to the requirements of CSA A3001 Type SF in a ratio of 6% to 8% of the mass of the cement.

4-4.18.2.5 Admixtures

A. Air entraining admixtures must conform to the requirements of ASTM C260.

B. Chemical admixtures such as water reducers, high-range water reducers (superplasticizers), and retarders, must conform to the requirements of ASTM C1141.

C. Admixture containing chlorides must not be used.

D. All admixtures and set accelerators must be sourced from a single manufacturer. The manufacturer must state compatibility between admixtures.

4-4.18.2.6 Aggregates

A. Fine and coarse aggregates must comply with CAN/CSA A23.1.

B. The relative density and absorption of fine and coarse aggregates must be determined in accordance with CSA A23.2-6A and CSA A23.2-12A.

C. The water absorption of the combined aggregate must not exceed 2%.

D. Use nominal 10 mm maximum size coarse aggregate combined with a concrete sand to provide a blend that conforms to Table 4-4.18.2.6 [*Composite Gradation Envelope*].

Table 4-4.18.2.6 Composite Gradation Envelope

Sieve Size (mm)	Total Passing each sieve % by mass
14	100
10	90-100
5	70-85
2.5	50-70
1.25	35-55
0.630	20-35
0.315	8-20

Sieve Size (mm)	Total Passing each sieve % by mass
0.160	2-10

4-4.18.2.7 Reinforcement

A. Concrete reinforcement must comply with Section 4-4.10 [*Concrete Reinforcement*] of this Schedule.

B. Mill test reports must be submitted for all steel reinforcement, including welded wire mesh.

C. Mill test reports originating from a mill outside of Canada or the United States of America must meet the requirements of Section 4-4.11.3.4 [*Mill Certificates*] of this Schedule.

4-4.18.2.8 Supply and Equipment

A. Wet-mix shotcrete must be batched, mixed, and supplied in accordance with the following.

   1. Central mixing with transit mixture delivery

      a. Aggregate, cement, and silica fume must be mass batched in a central mixer in accordance with the requirements of CSA A23.1-04.

      b. Water and chemical admixtures must be batched to the accuracy specified in CSA A23.1-04. Transit mixers must be free of excessive accumulations of hardened shotcrete or concrete in the drum or on the blades.

      c. Blades must be free of excessive wear.

      d. Transit mixture delivery must conform to the requirements of CSA A23.1-04. All shotcrete must be shot within 60 minutes after addition of mixture water to the batch. Shotcrete loads must be of such batch size that this requirement is met. This time limit may be extended, subject to acceptance by the Designer, if proper use is made of set retarding or hydration controlling admixtures to maintain workability without re-tempering with water.

   2. Transit mixing and delivery

      a. The same requirements as central mixing except that all mix constituents must be added directly to the transit mixer instead of the central mixer.

      b. Transit mixers must be filled to not more than 70% of their rated capacity, to enable efficient mixing action.

4-4.18.2.9 Shotcrete Placing Equipment

A. The shotcrete placing equipment must be capable of delivering a steady stream of uniformly mixed material to the discharge nozzle at the proper velocity and rate of discharge.

B. The use of positive displacement pumps equipped with hydraulic or mechanically powered pistons (for example, similar to conventional concrete piston pumps), with compressed air added at the discharge nozzle, is the preferred type of wet-mix shotcrete delivery system.

C. Pneumatic feed guns, rotary type feed guns (similar to dry-mix guns) and peristaltic squeeze-type pumps must only be used if the Design-Builder can demonstrate that they can produce shotcrete meeting the Project Requirements.

D. The air ring at the nozzle must be carefully monitored for any signs of blockage of individual air holes. If non-uniform discharge of shotcrete becomes apparent, shooting must be stopped, and the air ring cleaned or other appropriate corrective actions taken.

E. The delivery equipment must be thoroughly cleaned at the end of each shift. Any buildup of coatings in the delivery hose and nozzle must be removed. The air ring and nozzle must be regularly inspected and cleaned and replaced if required.

4-4.18.2.10 Auxiliary Shotcrete Equipment

A. Clean, dry, compressed air capable of maintaining sufficient nozzle velocity for all parts of the Work and simultaneous operation of a blowpipe must be supplied.

B. The air supply system must contain a moisture and oil trap to prevent contamination of the shotcrete.

C. It is the Design-Builder's responsibility to supply auxiliary shotcrete equipment such as material delivery hoses, blowpipes, and couplings as required to complete the Work.

4-4.18.2.11 Nozzleman Qualification

A. The nozzle operator must be an ACI-certified Shotcrete Nozzleman for vertical applications for the shotcrete process type used.

B. The names of the nozzle operators and proof of their qualifications must be provided to the City.

4-4.18.2.12 Alignment Control and Cover

A. Implement alignment control to establish control over line and grade and ensure that the minimum specified shotcrete thickness and cover to concrete reinforcement are maintained. Verify that reinforcing bars are fixed to provide specified cover before application of any shotcrete.

B. Provide alignment control by means of devices such as shooting wires, guide strips, depth gauges, or forms. Depth gauges must be installed at 1.8 m spacing longitudinally and transversely with no less than two gauges per increment of surface area to receive shotcrete.

C. When ground wires (also called guide wires or shooting wires) are used, they must consist of a high strength steel wire kept taut during shotcreting. Ground wires must be removed after completion of shotcreting and screeding operations.

D. Guide strips and forms must be of such dimensions and installation configuration that they do not impede the ability of the nozzlemen to produce uniform, dense, properly consolidated shotcrete. In particular, installations that are conducive to the entrapment of rebound or formation of shadows and voids must not be used.

4-4.18.2.13 Preconstruction Trial

A. The Design-Builder must carry out a preconstruction trial to prequalify the nozzlemen proposed for use on the project. Nozzlemen who have not been prequalified are not permitted to apply shotcrete on the project. The preconstruction trial must use the same materials, shotcrete mixture, and equipment proposed for use on the project and approximate actual working conditions, configuration, reinforcement, and shooting position, as near as practicable.

B. Nozzlemen must prequalify by shooting mock-ups of the reinforced structural wall element. Five cores must be taken from each mock-up for core grading from locations as determined by the Designer. Cores must be evaluated by the Designer to check the quality of shotcrete placement. Cores must show adequate consolidation and be free of excessive voids around concrete reinforcement, shadows, sags, sloughing, or delamination.

C. The Design-Builder must prepare and cure test panels according to ASTM C1140. Test panels must have a minimum length and width of 600 mm and 150 mm deep. The test panels must be made from wood and sealed plywood and have 45° sloped edges to permit rebound to escape and facilitate demoulding. The reinforcement must equal the densest configuration expected in the shotcrete batch.

D. Test panels must be cured in the field, close to the location where shot, for 2 days before being transported in the form to a testing laboratory. The test panels must be cured under wet burlap covered with plastic sheet under temperature conditions similar to that experienced by the wall. The panels must be protected from disturbance or damage.

E. Test panels and cores extracted from the test panels must be moist cured at 23°C and in accordance to AASHTO M201 until the time of compressive strength testing. Alternatively, the test panels and core samples must be covered and tightly wrapped with material conforming to ASTM C171.

F. After 14 days, but no later than 28 days after shooting, perform and report concrete quality tests including density, boiling absorption and volume of permeable voids. At least three samples must be tested from each non-reinforced test panel. Samples may consist of cores, pieces of cores or test panels that are without observable cracks, fissures, or shattered edges.

G. Three core samples must be drilled, 75 mm in diameter, from each test panel at least 40 hours prior to both the 7 day and 28-day compressive Strength Tests. The cores must be collected and tested in accordance with ASTM C1140. Before compressive strength testing, saw or tool the ends of the cores to eliminate projections and to achieve perpendicularity to the longitudinal axis. Compressive Strength Tests of the three cores in accordance with CSA A23.2-9C must be carried out at 28 days.

H. If the preconstruction test specimens fail to meet the Project Requirements, the materials, mix design and application must be adjusted and a new test panel must be shot. No Project Work must commence until the preconstruction requirements have been met.

I. If the source or quality of the materials or the mix proportions change, new shotcrete trials must be completed prior to using the shotcrete new mix design for production.

J. The Design-Builder must submit results from all shotcrete trials, including the following information:

1. test panel and core identification including panel number, shooting orientation, mix proportions and nozzle operator;
2. date and time of test panel application including dimensions, size and spacing of reinforcement, and type of curing;
3. date and time specimen was tested;
4. curing time for each specimen;
5. strength of each core specimen;
6. dimensions of each core specimen and sketch of each failed core specimen; and
7. measured strain at failure of each core specimen.

4-4.18.2.14 Construction Testing

A. The Design-Builder must submit a construction testing plan to the City before beginning shotcrete construction.

B. One construction test panel must be shot for each 50 m^3 of shotcrete production, or for each day of shotcrete production, whichever is more frequent. The panel must be shot in the same orientation as the work being done.

C. Construction test panels must be produced, stored, handled, cured, and tested in the same manner prescribed for preconstruction test panels.

D. All results from production tests must be provided to the City.

4-4.18.2.15 Shotcrete Application and Finishing

A. All shotcrete work must follow good industry practice as defined in ACI 506.

B. Concrete reinforcement must be supported so it is not displaced during the application of shotcrete.

C. Shotcrete must not be applied to frozen surfaces. The Design-Builder must dampen surfaces and confirm that the soil is free of surface water prior to shotcrete application.

D. Shotcrete application must be in layers no greater than 75 mm unless the Design-Builder can demonstrate that a thicker application can be achieved without sloughing or sagging. When applying more than one layer of shotcrete, trim with a cutting rod, or brush with a stiff bristle broom to remove all loose material, overspray, laitance, or other material detrimental to bonding of the next layer of shotcrete. Each shotcrete layer must be allowed to stiffen sufficiently before applying next layer of shotcrete. If shotcrete has set and hardened, high pressure clean water (34.4 MPa) must be used to blast the surface. The surface must then be soaked for 2 to 24 hours and excess water must be blown away immediately prior to placing the next layer of shotcrete to provide a saturated surface dry condition.

E. The Design-Builder must use a shooting technique that provides full encapsulation of all concrete reinforcement and embedments. All voids, shadows, sags, and/or other defects must be cut out from the applied shotcrete while it still plastic and re-shot.

F. The shotcrete must be trimmed with a cutting rod or other suitable device to the specified line and grade. The shotcrete must be finished to a sandy texture acceptable to the Designer. The Design-Builder must protect all fixtures and adjacent concrete surfaces from buildup of rebound, overspray and shotcrete trimmings, and promptly remove any excess shotcrete applied outside of the specified areas to be shot.

G. Construction joints must have a 45 degree tapered edge. Square construction joints are not permitted. The shotcrete must be cut while plastic with a trowel or other suitable tool to form construction joints. The shotcrete must be green cut with a 34.4 MPa water pressure jet the following day, if necessary, to remove loose material. Feather-edge construction joints are not permitted.

H. The general requirements for hot and cold weather concreting detailed in CSA A23-1 must apply to the shotcrete work. Shotcrete application must be terminated if the ambient temperature rises above 30°C, unless the Design-Builder adopts special hot weather shotcreting procedures acceptable to the Designer.

I. Shotcrete must not be applied during high winds or heavy rainfall. The shotcrete mix temperature must be maintained between 10°C and 30°C.

J. During periods of cold weather, shotcreting may only proceed if the substrate to which the shotcrete is applied is above 5°C for a minimum of 24 hours prior to application.

K. For shotcreting of the final excavation face, the application of the shotcrete must not be delayed by more than 8 hours without acceptance from the Designer.

4-4.18.2.16 Shotcrete Facing Tolerances

A. The shotcrete facing must meet the tolerances listed in Table 4-4.18.2.16 [*Shotcrete Facing Tolerances*].

Table 4-4.18.2.16 Shotcrete Facing Tolerances

Item	Tolerance
Horizontal location of welded wire mesh, reinforcing bars, and headed studs	10 mm
Thickness of shotcrete	15 mm
Planeness of finish face surface, gap under 3 m straightedge	15 mm
Nail head bearing plate deviation from parallel to wall face	5 degrees

4-4.18.2.17 Shotcrete Surface Finish

A. A smoothing layer a minimum of 25 mm thick must be applied to create a smooth surface for the installation of the frost protection insulation.

B. The shotcrete must not have any irregularities that exceed a ratio of five units of length to one unit of depth, and its minimum radius must be 200 mm.

C. The final surface must be free from structural steel, fixings, and any other sharp edges or pointed forms.

4-4.18.2.18 Curing

A. Shotcrete must be moist cured using fogging, wetting or maintenance of a minimum 95% relative humidity in the area surrounding the shotcrete, for a minimum of 7 days. Moist curing must be accomplished using one or more of the following procedures:

   1. wrap the elements in wet burlap covered with a plastic sheet or a pre-saturated plastic coated non-woven synthetic fibre; or

   2. install sprinklers, soaker hoses, or other devices to keep the shotcrete continuously wet for the specified period.

B. The use of intermittent wetting procedures that will allow the shotcrete to undergo cycles of wetting and drying during the curing process is not permitted.

C. If the prevailing ambient conditions (relative humidity, wind speed and air temperature) are such that the shotcrete develops plastic shrinkage and/or early drying shrinkage-cracking, terminate shotcrete application.

D. Corrective measures such as the installation of wind barriers or fogging devices to protect the work must be implemented before restarting shotcrete application. Do not proceed with shotcrete application if the rate of evaporation at the shotcrete surface exceeds 1.0 kg/m^2/hr as detailed in CSA A23.1-04, Appendix D.

E. After application of the shotcrete, the air temperature at the shotcrete surfaces must be maintained at 10°C or greater for at least 4 days after the application of shotcrete. The means of maintaining the air temperature must be acceptable to the Designer. The use of unvented heaters that give rise to carbonation are prohibited.

4-4.19 DEMOLITION AND DISPOSAL

4-4.19.1 General

A. This specification is for the demolition and disposal of existing concrete structures for the 111 Street Roadway Bridge Widening. This Project Work must be completed in accordance with the Agreement and as determined by the City.

B. Demolished bridge structure materials are the property of the Design-Builder.

4-4.19.2 Demolition and Disposal

4-4.19.2.1 General

A. The Design-Builder must remove the demolished materials in accordance with the Agreement, these specifications and as determined by the City.

B. The Design-Builder must submit a detailed plan for the demolition and correct disposal of materials from the bridge to the City for review and acceptance a minimum of 4 weeks prior to commencement of the Project Work. The demolition and disposal plan must include drawings and supporting documents necessary to describe the following:

1. TAS;
2. access to work;
3. temporary works and support structures;
4. temporary stabilization details and measures;
5. type and capacity of equipment;
6. sequence of operation including position of equipment and cranes;
7. position of equipment and cranes relative to substructure elements such as abutment backwalls, with details of load distribution of wheels and outriggers;
8. lifting devices and lifting points showing lifting forces;
9. demolition sequence;
10. revisions to the ECO plan to address all environmental regulations and Agreement requirements; and
11. proposed disposal procedures and disposal locations.

C. The demolition and disposal plan will be considered a Professional Work Product and must be authenticated by a Professional Engineer licensed to practice in the Province of Alberta and validated by a Responsible Member, in accordance with APEGA requirements.

D. No demolished debris must be permitted to fall into Blackmud Creek.

E. After the demolition and disposal plan has been reviewed and accepted by the City a milestone construction meeting must occur a minimum of 1 week prior to commencement of the Project Work. The Design-Builder's project manager, field superintendent, and any specialty subcontractors must attend the construction milestone meeting at date, time and location acceptable to the City. A construction milestone meeting will not be required for the demolition, disposal and salvage of culverts.

F. Safety and compliance with the Occupational Health and Safety Act and Regulations thereunder, must be an integral part of the design of this Project Work. The Design-Builder must immediately provide the City with any documentation required by Occupational Health and Safety Act and Regulations upon request.

G. The Design-Builder must continue to be fully responsible for the results obtained by the use of demolition and disposal plan, with the Professional Engineer also assuming responsibility, as the Design-Builder's Agent, for the results obtained.

H. The City's review and acceptance must not be considered as relieving the Design-Builder of the responsibility for the safety of his methods or equipment, nor from completing the Project Work in accordance with his demolition and disposal plan and these specifications. The Design-Builder must not commence any Work until review and acceptance of the demolition and disposal plan by the City has been obtained.

4-4.20 STAY CABLES & HANGERS

4-4.20.1 General

A. This Section 4-4.20 [*Stay Cables & Hangers*] sets out requirements for all stay cables and hangers forming part of a Structure including minimum requirements for the supply, fabrication, delivery and installation of stay cables.

   1. Stay cables and hangers include main tensile elements (strands), strand sheathing, anchorage assemblies, saddles, damping devices, anti-vandalism tubes and corrosion protection provisions.

B. References in PTI DC45.1 to the terms "Professional Engineer" must be taken to mean "Designer".

   1. All submissions made to the "Designer" must also be made to the City.

4-4.20.2 Engineering Data

A. Shop drawings, procedures and calculations must be submitted to the City. The Review Period must be 30 Business Days.

B. Any subsequent deviation from methods, materials, or details will not be permitted unless the affected submittals are updated in advance of use.

C. Shop drawings must be prepared showing all dimensions, materials and operations for fabrication, assembly and installation of the stay cables & hangers.

D. Detailed step by step procedures must be provided for installing all stay cable and hanger components and for stressing, adjusting the force in and de-stressing the stay cables and hangers. The procedures must be consistent with the actual loads, sequences, schedules, material properties, cable forces and all other aspects of the Construction of the Structure. These procedures must include a plan for control of the geometry of the bridge during stressing to ensure the final profile meets the design requirements.

B. Supporting calculations must be prepared showing the forces and elongations required for each stay cable and hanger.

C. All submittals must be authenticated by a Professional Engineer.

4-4.20.3 Materials

4-4.20.3.1 Individual Strand Sheathing

A. The sheathing around the individual stay cable and hanger strands must be HDPE sheathing and must comply with the requirements of PTI DC 45.1, Section 3.3.

4-4.20.3.2 Corrosion Inhibiting Coating

A. The intermediate spaces between the strand wires and between the wires and the individual sheath of each stay cable and hanger strand must be filled with a corrosion inhibiting coating meeting the requirements of PTI DC45.1, Section 3.3.

   1. The filler must be continuous and durably stable.

4-4.20.3.3 Outer Stay Pipe

A. The outer stay or hanger pipe must be a HDPE pipe and as a minimum must comply with the requirements of PTI DC45.1, Section 3.5.

B. In addition, the outer stay or hanger pipe must meet strength and durability requirements under all anticipated cold weather conditions.

4-4.20.4 Fabrication

4-4.20.4.1 General

A. The fabrication of stay cables and hangers must comply with Section 6.0 of PTI DC45.1.

B. The fabrication of stay cables and hangers must conform to Section 4-4.11 [*Structural Steel*] of this Schedule and to the AASHTO LRFD BCS where applicable.

4-4.20.4.2 Stay Cables & Hangers

A. Stay cable components must be protected from corrosion, heat, abrasion and other harmful effects throughout fabrication, shipping, delivery, storage and installation.

B. All damage to stay cables and hangers or components thereof must be evaluated and remedied prior to installation of the stays and hangers.

   1. Any damaged strand or corrosion protection barrier must be replaced.

4-4.20.4.3 Strand

A. During the process of manufacture of individual wires for “weldless” strand, welding must be permitted only prior to or at the site of the last thermal treatment of the wire, e.g., patenting or controlled cooling.

   1. There must be no welds in the wire after it has been drawn through the first die in the wire drawing process.

B. Strand must be supplied in coils and each strand coil must be protected with a manufacturer approved method so that the strand has no adhering foreign matter or damage to the corrosion protective coating, including that from ultraviolet exposure.

   1. The ends of the strands must be sealed to prevent intrusion of moisture into the annular space between the wires.

2. When uncoiled the strand must lay straight with a maximum deviation of 100 mm from a theoretical centreline in any 1800 mm length.

3. Handling of the strands must not result in sharp kinks or short radius bends less than the coil radius.

4. The strand coils must be supported on wooden or steel reels with padded contact areas, wherever practicable.

5. Upon delivery, the strand must be stored in a weatherproof enclosure to prevent corrosion or other damage.

C. Welds or joints are not permitted in the finished strand.

D. Each strand coil must be marked with the order number, coil number, heat number and the starting end.

E. Individual strands comprising the stay cable or hanger must be cut to their precalculated lengths while fully supported on a rigid calibrated cutting bed.

2. Strands must be cut using abrasive saws or shears.

3. Flame cutting is not permitted.

4-4.20.4.4 Outer Stay Pipe Fusion Welds

A. When the outer stay or hanger pipe is made up of sections assembled end-to-end, the sections must be assembled by butt fusion welding in accordance with ASTM D2657 (Hot-Plate Welding).

1. The cut ends of sections of the joined pipe must be perpendicular to the pipe axis.

2. The pipe welds must be flush with the pipe inside and outside.

3. Welding of the pipe when the stay strands are in the pipe is not permitted.

B. The welds must be performed on a test section of pipe for the cable cross section as shown on the applicable final Design Data.

1. The necessary tests must be performed to ensure that the weld develops the required strength and that the joint is hydrostatically sealed.

4-4.20.5 Testing and Qualification

A. Testing and qualification of the stay cables and hangers, including strands, strand sheathing, anchorage assemblies, saddles and corrosion protection provisions must comply with the requirements of PTI DC45.1.

4-4.20.5.1 Anchorage Testing

A. Testing of the stay cable and hanger anchorages must be carried out in accordance with the AASHTO LRFD BCS, Section 10.3.2.3.

1. The crack width requirements of AASHTO LRFD BCS Section 10.3.2.3 must be met.

4-4.20.6 Installation

4-4.20.6.1 General

A. Stay cables and hangers must be installed in a manner that does not damage the stay cables and hangers including strands, strand sheathing, anchorages assemblies, saddles and corrosion protection.

1. Stay cables and hangers must be stored, handled, transported, assembled and installed in accordance with the procedures prescribed by the stay cable supplier.

2. The installation of stay cables and hangers must be in accordance with Section 6.0 of PTI DC 45.1.

3. During installation the leading ends of the strands must be prevented from damaging the stay pipe or the sheaths of the strands installed previously.

4. The stay cable and hanger anchorages, saddles, guidance systems, etc. must be fitted with protection to avoid damaging the individual protection of strands during installation.

5. The strands must be parallel throughout their length and must pass through matching holes in the anchorage heads.

B. Stay cables and hangers, anchorages, and guide systems must be installed and aligned based on a geometric control plan that accounts for changes in Structure geometry after their installation.

4-4.20.6.2 Stressing

A. Stay cables and hangers must be stressed in accordance with the procedures prescribed by the stay cable or hanger supplier and so as to produce the forces in the cables shown on the applicable final Design Data.

1. Stay cable and hanger strands may be tensioned one-by-one provided that it can be demonstrated that the final tension and elongation of each strand is equalized within a range of ± 2.5% GUTS.

2. All stressing results must be recorded automatically with a computer system connected to load cells.

B. Jacks and gauges for stay cable and hanger installation must be calibrated with reference to a standard pressure gauge or load cell within one month prior to the beginning of cable installation, and every 6 months thereafter, or for 500 strands maximum, for the duration of cable installation.

1. The standard gauge must have a relative accuracy of at least ± 0.5%.

C. Stay cables and hangers must be capable of being tensioned, de-tensioned and re-tensioned more than once during the Construction of the Structure.

1. These operations may be carried out either by full jacking of the live anchorage or strand by strand.

2. Stay cables and hangers must not be de-tensioned in such a way that during de-tensioning the "gripping" zone is incorporated in the stressed portion of the stay. The "gripped" zone must not be left permanently on the stressed portion of the stay.

4-4.20.6.3 Monitoring & Adjustment

A. Stay cables and hangers must be adjusted in the final dead load condition such that each individual cable does not deviate from the stay cable dead load force values shown on the applicable final Design Data by more than ± 5%.

   1. Force verification must be performed with full head lift off tests.

B. Strands extending through the stay cable saddles must be monitored for any possible slippage during Construction and adjusted if needed in accordance with procedures prescribed by the stay cable supplier.

4-4.20.6.4 Documentation

A. Permanent records of the stay cable and hanger installation must be prepared in accordance with the requirements of PTI DC45.1, Section 6.9.6.2.

   1. The permanent records must include the As-Built deck profiles and final cable loads that constitute the base line survey.

4-4.21 PRECAST CONCRETE PARTIAL DEPTH DECK PANELS

4-4.21.1 General

A. This Section 4-4.21 [*Precast Concrete Partial Depth Deck Panels*] sets out the requirements for all deck on girder slab using precast concrete partial depth deck panels forming part of a Structure including minimum requirements for the supply, manufacture, delivery and erection.

B. Requirements for cast-in-place concrete are given in Section 4-4.6 [*Cast-In-Place Concrete*] of this Schedule.

C. Requirements for precast concrete and pre-tensioning are given in Section 4-4.7 [*Precast Concrete*] of this Schedule.

4-4.21.2 Supply and Fabrication Standards

A. The panels must be cast flat.

B. All edges of the panel must have a minimum 20 x 20 mm chamfer, except the transverse joint which must have a 55x55 mm chamfer along the top edges.

C. Panel identification tags cast into the surface of the precast concrete partial depth deck panels are not permitted. Panel identification methods must be acceptable to the City.

D. All pre-tensioning strands must be cut flush with the precast concrete partial depth deck panel edges, and the ends of the pre-tensioning strands must be sealed with Sikadur-31 or an approved equivalent.

E. The top surface of precast concrete partial depth deck panels must be clean, free of laitance, and roughened to 3 mm amplitude with spacing not greater than 15 mm with grooves parallel to strands. Formed chamfer surfaces that will be in contact with cast-in-place concrete must be sandblasted to remove all laitance and uniformly expose aggregate particles.

F. Precast concrete partial depth deck panels must meet the following tolerances:

   1. Panel lengths: ± 5 mm (as measured perpendicular to the girder lines).

2. Panel widths: ± 10 mm (as measured parallel to the girder lines).
3. The maximum difference in plan view diagonal dimensions (squareness) of rectangular panels must not be greater than 3.5 mm per metre of diagonal length.
4. Thickness of panel: + 5 mm, - 3 mm.
5. Pre-tensioning strands must be located at the centroid of the panel with a vertical tolerance of +0 mm, - 3 mm, measured from the soffit and a horizontal tolerance of ± 10 mm.
6. Deviation from straightness of panel edges along the transverse joint between adjacent panels must not exceed 1.5 mm per metre length.
7. Vertical bowing of panels out of plane, after casting and immediately prior to erection, in the direction of measurement, must not be greater than the panel length/360 or the panel width/360, whichever is less, and must in no case exceed 10 mm maximum.
8. Warping of panels must not be greater than 5 mm per metre of distance from the nearest adjacent corner.

G. Tolerance measurement results must be provided to the City forthwith, upon request. Panels not meeting any of the tolerances listed above will be rejected.

H. A precast concrete partial depth deck panel having any one of the following defects or deficiencies will be rejected:

1. Panels with honeycombing or spalls when the depth exceeds 15 mm or when the area of defect exceeds 25 mm x 25 mm.
2. Panels with any voids or spalls in the bottom of the panel.
3. Panels with any crack located parallel to or over the pre-tensioning strands or reinforcing steel.
4. Panels with any crack at the edges and / or with cracks at the bottom.
5. Panels with cracks that are deeper than 25 mm and/or wider than 0.1 mm.

4-4.21.3 Erection and Construction Standards

A. The precast concrete partial depth deck panels must be erected on the girders with temporary supports. The precast concrete partial depth deck panels must be erected so that the transverse joints between adjacent panels are never greater than 5 mm. All transverse joints must be sealed with Sikaflex 15LM or an approved equivalent to prevent mortar leakage.

B. The Design-Builder must survey all girders at locations corresponding with those detailed on the camber diagram and determine girder haunch dimensions required to achieve design grades. The City must be afforded full and safe access for any independent haunch dimensional check to confirm deck thickness and grades prior to deck reinforcing steel placement. In the event that actual girder camber values vary significantly from the estimated values indicated on the Design Drawings, the Design-Builder may raise or lower the grades when accepted by the City.

C. All precast concrete partial depth deck panel system formwork drawings and design notes will be considered Professional Work Products and must be authenticated by a Professional Engineer licensed to practice in the Province of Alberta and validated by a Responsible Member, in accordance with APEGA requirements.

D. Additionally, the system must be inspected prior to placing concrete to confirm conformance with the Design Drawings. The Design-Builder must design and install support brackets such that no damage to girder flanges and webs will result. Where brackets bear against girder webs, the Design-Builder must protect the contact surface with timber or neoprene softeners. No drilling of additional holes, or any other modifications including field welding, made to the superstructure elements is permitted. Effects of concentrated loads on thin webs must be checked, and where necessary, sufficient means must be provided to distribute or carry such concentrated loads to the supporting flanges or stiffeners. Formwork for deck overhangs, curbs, sidewalks and parapets must be fabricated so that the lines and grades shown on the Design Drawings are achieved, with adjustments made where necessary to compensate for variances in girder dimensions, positioning, alignment and sweep. All lifting hooks and deck panel levelling bolts must be cut flush with the top of the deck panel after the profile, deck concrete thickness and girder haunch dimensions have been completed, checked and accepted by the City, and before reinforcing steel placement.

E. The haunches must be formed to be flush with the edge of the girder flanges. Formwork must be sealed against girder flanges such that concrete paste leakage does not occur. All haunch forming material, including sealants, must be completely removed after casting the deck to fully expose the haunch concrete.

F. It is not permitted for any hardware associated with deck formwork, including deck overhang formwork, to be visible after removal of all formwork. For precast concrete girder superstructures, anchors for the exterior deck overhang formwork may be cast into the girder top flanges above the web. For steel girder superstructures, anchors for the exterior deck overhang formwork may be shop attached to the girder top flanges. Field welding or drilling of the girders or precast concrete partial depth deck panels is not permitted.

G. Prior to the placement of deck reinforcing steel and prior to the placement of deck concrete, the surfaces of precast concrete partial depth deck panels, girder flanges, and all formwork must be thoroughly cleaned with high pressure water. Cleaning must be completed in a controlled and progressive manner from the high to low ends of the deck pour area in both transverse and longitudinal directions. Appropriate wash water drains must be incorporated into haunch and bulkhead formwork. All surfaces must be free of dirt, debris or foreign materials. All hardened concrete surfaces to receive deck concrete must be brought to and kept in a saturated surface dry condition, free of standing water, a minimum of 2 hours prior to concrete placement. The deck and haunch concrete must be cast monolithically in a two stage process to ensure full consolidation of concrete in the haunch area.

H. The first stage must include placement of concrete in the haunch area and over the girder top flange in continuous strips. The depth of the first stage pour must be above the bottom surface of the precast concrete partial depth deck panel, but must not exceed the top surface of the precast concrete partial depth deck panel, and must not extend in front of the second stage pour by more than 6 m. Placement and consolidation of concrete in the first stage must be completed in such a manner that entrapped air on the vertical and horizontal formed surfaces of the haunch is minimized.

I. The second stage must include placement of the remaining deck concrete. Concrete placement must occur in a timely manner as to not result in any cold joint between the first and second stages. If cold joints are produced, the entire deck section must be removed and replaced including but not limited to the cast-in-place HPC concrete, steel reinforcing bars and precast concrete partial depth deck panels.

J. Voids, cavities, or areas of honeycombing found in the haunch concrete meeting the following parameters must be repaired by the Design-Builder:

    1. Any defects with depth greater than or equal to 20 mm.

    2. Defects greater than or equal to 25 mm high or 25 mm wide x 10 mm deep.

3. 10 or more defects between 20 mm wide or 20 mm high x 15 mm deep per lineal metre.

4. 30 or more defects between 10 mm wide or 10 mm high x 15 mm deep per lineal metre.

K. Proposed repair procedures must be submitted for review and acceptance by the City.

Appendix 4-1.2A: 23 Avenue Underpass – Geotechnical Baseline Conditions

Appendix 4-1.5A: Anthony Henday Drive LRT Bridge Preliminary Design Drawings