Division 6 Structures

6-01 General Requirements for Structures

6-01.1 Description

This section relates to structural and incidental items used in any or all types of existing or proposed Structures. These provisions supplement the detailed Specifications supplied for a given Structure. These provisions apply only when relevant and when they do not conflict with the Plans or Special Provisions.

6-01.2 Foundation Data

Foundation data in the Plans (from test borings, test pits, or other sources) were obtained only to guide the Department in planning and designing the project. These data reasonably represent the best information available to the Department concerning conditions and materials at the test sites at the time the investigations were made.

6-01.3 Clearing the Site

The Contractor shall clear the entire site of the proposed Structure to the limits staked by the Engineer.

6-01.4 Appearance of Structures

To achieve a more pleasing appearance, the Engineer may require the Contractor to adjust the height and alignment of bridge railings, traffic barrier, and structural curbs.

6-01.5 Vacant

6-01.6 Load Restrictions on Bridges Under Construction

Bridges under construction shall remain closed to all traffic, construction equipment and material storage until the Substructure and the Superstructure, through the bridge deck, are complete for the entire Structure, except as provided herein. Completion includes release of all falsework, removal of all forms, and attainment of the minimum design concrete strength and specified age of the concrete in accordance with these Specifications. Once the Structure is complete, Section 1-07.7 shall govern all traffic loading, including vehicle traffic and construction equipment.

The Contractor may only store material on a bridge span under construction that will become part of that bridge span. The material shall not be stored within the middle third of the span. At the request of the Engineer, the Contractor shall provide supporting documentation of all material loads.

If necessary and safe to do so, and if the Contractor requests it through a Type 2E Working Drawing, the Engineer may allow traffic, construction equipment and material loads (in addition to those defined above) on a bridge prior to completion for loads and reaction locations not identified in the Plans. The written request shall:

1. Describe the extent of the Structure completion at time of the proposed equipment loading;
2. Describe the loading distribution, magnitude, arrangement, movement, and position of all traffic, construction equipment, and materials on the bridge, including but not limited to the following:
    a. Locations of all construction equipment, including outriggers, spreader beams and supports for each, relative to the bridge framing plan (bridge girder layout);
    b. Mechanism of all load transfer (load path) to the bridge;

3. Provide calculations under the design criteria specified in the current AASHTO *LRFD Bridge Design Specifications*, including interims, and the current WSDOT *Bridge Design Manual LRFD* M 23-50, including at a minimum the following calculations due to traffic or construction loads:
    a. Factored demands and capacities using the Strength I Limit State in the main load carrying members and bridge deck for all construction loading on the bridge;
    b. Factored demands and capacities at the Strength IV Limit State in the main load carrying members and bridge deck for all material loads stored on the bridge;
    c. Stresses in prestressed concrete members at the Service I, II and III Limit States as applicable, including the allowable stress limits specified in Section 6-02.3(25)L3;
    d. Construction deflections at the Service I Limit State, where required by the Engineer.
4. Provide supporting material properties, catalogue cuts, and other information describing the construction equipment and all associated outriggers, spreader beams, and supports; and
5. State that the Contractor assumes all risk for damage.

6-01.7 Navigable Streams

The Contractor shall keep navigable streams clear so that water traffic may pass safely, providing and maintaining all lights and signals required by the U.S. Coast Guard. The Contractor shall also comply with all channel depth and clearance line requirements of the U.S. Corps of Engineers. This may require removing material deposited in the channel during construction.

6-01.8 Approaches to Movable Spans

No bridge deck or sidewalk slab on the approach span at either end of a movable span may be placed until after the movable span has been completed, adjusted and closed.

6-01.9 Working Drawings

All Working Drawings required for bridges and other Structures shall conform to Section 1-05.3.

6-01.10 Utilities Supported by or Attached to Bridges

Installation of utility pipes and conduit systems shall conform to the details shown in the Plans and as specified in the utility agreement between the utility company and the Contracting Agency.

All utility pipes and conduit systems supported by or attached to bridges shall be labeled with reflective sheeting conforming to Section 9-28.12, and the following:

Content	Label Background Color	Lettering Utility Color
Electrical Power	Red	Black
Gas, Oil, Steam, Petroleum, and other gaseous materials	Yellow	Black
CATV, Telecommunication, Alarm, and Signal	Orange	Black
Potable Water	Blue	White
Reclaimed Water, Irrigation, Slurry	Purple	White
Sewer and Storm Drain	Green	White

The purple color background for the label for reclaimed water, irrigation, and slurry, shall be generated by placing transparent film over white reflective material. The purple tint of the transparent film shall match SAE AMS Standard 595, Color No. 37100.

The label text shall identify the utility contents and include the One-Number Locator Service phone number 1-800-424-5555.

The minimum length of the label color field shall be the longer of either 1 letter width beyond each end of the label text, or the length specified below:

Minimum Pipe O.D. (inches)	Maximum Pipe O.D. (inches)	Minimum Length of Label Color Field (inches)	Letter Height (inches)
¾	1¼	8	½
1½	2	8	¾
2½	6	12	1¼
8	10	24	2½
12		32	3½

Utility pipes and conduit systems shall be labeled on both sides of each bridge pier, and adjacent to each entrance hatch into a box girder cell. For utility pipes and conduit systems within bridge spans exceeding 300 feet, labels shall also be applied to the utility pipes and conduit systems between the piers at a maximum spacing of 300 feet. The label shall be visible at a normal eye height.

6-01.11 Name Plates

The Contractor shall install no permanent plates or markers on a Structure unless the Plans show it.

6-01.12 Final Cleanup

When the Structure is completed, the Contractor shall leave it and the entire site in a clean and orderly condition. Structure decks shall be clean. Temporary buildings, falsework, piling, lumber, equipment, and debris shall be removed. The Contractor shall level and fine grade all excavated material not used for backfill, and shall fine grade all slopes and around all piers, bents, and abutments.

6-01.13 Vacant

6-01.14 Premolded Joint Filler

When the Plans call for premolded joint filler, the Contractor shall fasten it with galvanized wire nails to 1 side of the joint. The nails must be no more than 6 inches apart and shall be 1½ inches from the edges over the entire joint area. The nails shall be at least 1½ inches longer than the thickness of the filler.

The Contractor may substitute for the nails any adhesive acceptable to the Engineer. This adhesive, however, shall be compatible with the material specified in Section 9-04.1(2) and capable of bonding the filler to portland cement concrete.

6-01.15 Normal Temperature

Bridge Plans state dimensions at a normal temperature of 64°F. Unless otherwise noted, these dimensions are horizontal or vertical.

6-01.16 Repair of Defective Work

6-01.16(1) General

When using repair procedures that are described elsewhere in the Contract Documents, the Working Drawing submittal requirements of this section shall not apply to those repairs unless noted otherwise.

Repair procedures for defective Work shall be submitted as Type 2 Working Drawings. Type 2E Working Drawings shall be submitted when required by the Engineer. As an alternative to submitting Type 2 or 2E Working Drawings, defective Work within the limits of applicability of a pre-approved repair procedure may be repaired using that procedure. Repairs using a pre-approved repair procedure shall be submitted as a Type 1 Working Drawing.

Pre-approved repair procedures shall consist of the following:

- The procedures listed in Section 6-01.16(2)
- For precast concrete, repair procedures in the annual plant approval process documents that have been approved for use by the Contracting Agency.

All Working Drawings for repair procedures shall include:

- A description of the defective Work including location, extent and pictures
- Materials to be used in the repair. Repairs using manufactured products shall include written manufacturer recommendations for intended uses of the product, surface preparation, mixing, aggregate extension (if applicable), ambient and surface temperature limits, placement methods, finishing and curing.
- Construction procedures
- Plan details of the area to be repaired
- Calculations for Type 2E Working Drawings

Material manufacturer's instructions and recommendations shall supersede any conflicting requirements in pre-approved repair procedures.

The Engineer shall be notified prior to performing all repair procedures and shall be given an opportunity to inspect the repair work being performed.

6-01.16(2) Pre-Approved Repair Procedures

6-01.16(2)A Concrete Spalls and Poor Consolidation (Rock Pockets, Honeycombs, Voids, Etc.)

This repair shall be limited to the following areas:

- Areas that are not on top Roadway surfaces (with or without an overlay) including but not limited to concrete bridge decks, bridge approach slabs or cement concrete pavement
- Areas that are not underwater
- Areas that are not on precast barrier, except for the bottom 4 inches (but not to exceed 1 inch above blockouts)
- Areas that do not affect structural adequacy as determined by the Engineer.

The repair procedure is as follows:

1. Remove all loose and unsound concrete. Impact breakers shall not exceed 15 pounds in weight when removing concrete adjacent to reinforcement or other embedments and shall not exceed 30 pounds in weight otherwise. Operate impact breakers at angles less than 45 degrees as measured from the surface of the concrete to the tool and moving away from the edge of the defective Work. Concrete shall be completely removed from exposed surfaces of existing steel reinforcing bars. If half or more of the circumference of a steel reinforcing bar is exposed, if the reinforcing bar is loose or if the bond to existing concrete is poor then concrete shall be removed at least ¾ inch behind the reinforcing bar. Do not damage any existing reinforcement.

Stop work and allow the Engineer to inspect the repair area after removing all loose and unsound concrete. Submit a modified repair procedure when required by the Engineer.

2. Square the edges of the repair area by cutting an edge perpendicular to the concrete surface around the repair area. The geometry of the repair perimeter shall minimize the edge length and shall be rectangular with perpendicular edges, avoiding reentrant corners. The depth of the cut shall be a minimum of ¾ inch, but shall be reduced if necessary to avoid damaging the reinforcement. For repairs on vertical surfaces, the top edge shall slope up toward the front at a 1-vertical-to-3-horizontal slope.
3. Remove concrete within the repair area to a depth at least matching the cut depth at the edges. Large variations in the depth of removal within short distances shall be avoided. Roughen the concrete surface. The concrete surface should be roughened to at least Concrete Surface Profile (CSP) 5 in accordance with ICRI Guideline No. 310.2R, unless a different CSP is recommended by the patching material manufacturer.
4. Inspect the concrete repair surface for delaminations, debonding, microcracking and voids using hammer tapping or a chain drag. Remove all additional loose or unsound concrete in accordance with steps 1 through 3.
5. Select a patching material in accordance with Section 9-20.2 that is appropriate for the repair location and thickness. The concrete patching material shall be pumpable or self-consolidating as required for the type of placement that suits the repair. The patching material shall have a minimum compressive strength at least equal to the specified compressive strength of the concrete.
6. Prepare the concrete surface and reinforcing steel in accordance with the patching material manufacturer's recommendations. At a minimum, clean the concrete surfaces (including perimeter edges) and reinforcing steel using oil-free abrasive blasting or high-pressure (minimum 5,000 psi) water blasting. All dirt, dust, loose particles, rust, laitance, oil, film, microcracked/bruised concrete or foreign material of any sort shall be removed. Damage to the epoxy coating on steel reinforcing bars shall be repaired in accordance with Section 6-02.3(24)H.
7. Construct forms if necessary, such as for patching vertical or overhead surfaces or where patching extends to the edge or corner of a placement.
8. When recommended by the patching material manufacturer, saturate the concrete in the repair area and remove free water at the concrete surface to obtain a saturated surface dry (SSD) substrate. When recommended by the patching material manufacturer, apply a primer, scrub coat or bonding agent to the existing surfaces. Epoxy bonding agents, if used, shall be Type II or Type V in accordance with Section 9-26.1.
9. Place and consolidate the patching material in accordance with the manufacturer's recommendations. Work the material firmly into all surfaces of the repair area with sufficient pressure to achieve proper bond to the concrete.
10. The patching material shall be textured, cured and finished in accordance with the patching material manufacturer's recommendations and/or the requirements for the repaired component. Protect the newly placed patch from vibration in accordance with Section 6-02.3(6)D.
11. When the completed repair does not match the existing concrete color and will be visible to the public, a sand and cement mixture that is color matched to the existing concrete shall be rubbed, brushed, or applied to the surface of the patching material and the concrete.

6-02 Concrete Structures

6-02.1 Description

This Work consists of the construction of all Structures (and their parts) made of portland cement or blended hydraulic cement concrete with or without reinforcement, including bridge approach slabs. Any part of a Structure to be made of other materials shall be built as these Specifications require elsewhere.

6-02.2 Materials

Materials shall meet the requirements of the following sections:

Cement	9-01
Aggregates for Concrete	9-03.1
Gravel Backfill	9-03.12
Joint and Crack Sealing Materials	9-04
Threaded Anchor Rods, Nuts, and Resin Bonding Material	9-06.4
Strip Seal Expansion Joint Components	9-06.19(1)
Modular Expansion Joint Components	9-06.19(2)
Reinforcing Steel	9-07
Epoxy-Coated Reinforcing Steel	9-07
Pigmented Sealer Materials	9-08.3(1)
Exposed Aggregate Concrete Coatings and Sealers	9-08.3(2)
Permeon Treatment	9-08.3(3)
Grout	9-20.3
Mortar	9-20.4
Curing Materials and Admixtures	9-23
Fly Ash	9-23.9
Ground Granulated Blast Furnace Slag	9-23.10
Microsilica Fume	9-23.11
Plastic Waterstop	9-24
Water	9-25
Fabricated Bridge Bearing Assemblies	9-31

6-02.3 Construction Requirements

6-02.3(1) Classification of Structural Concrete

The class of concrete to be used shall be as noted in the Plans and these Specifications. The class includes the specified minimum compressive strength in psi at 28 days (numerical class) and may include a letter suffix to denote structural concrete for a specific use. Letter suffixes include A for bridge approach slabs, D for bridge decks, P for piling and shafts, and W for underwater. The numerical class without a letter suffix denotes structural concrete for general purposes.

Concrete of a numerical class greater than 4000 shall conform to the requirements specified for either Class 4000 (if general-purpose) or for the appropriate Class 4000 with a letter suffix, as follows:

1. Mix design and proportioning specified in Sections 6-02.3(2), 6-02.3(2)A and 6-02.3(2)A1.
2. Consistency requirements specified in Section 6-02.3(4)C.
3. Temperature and time for placement requirements specified in Section 6-02.3(4)D.
4. Curing requirements specified in Section 6-02.3(11).

The Contractor may request, in writing, permission to use a different class of concrete with either the same or a higher compressive strength than specified. The substitute concrete shall be evaluated for acceptance based on the specified class of concrete. The Engineer will respond in writing. The Contractor shall bear all added costs that result from the change.

6-02.3(2) Proportioning Materials

The soluble chloride ion content shall be determined by the concrete supplier and included with the mix design. The soluble chloride ion content shall be determined by (1) testing mixed concrete cured at least 28 days or (2) totaled from tests of individual concrete ingredients (cement, aggregate, admixtures, water, fly ash, ground granulated blast furnace slag, and other supplementary cementing materials). Chloride ion limits for admixtures and water are provided in Sections 9-23 and 9-25. Soluble chloride ion limits for mixed concrete shall not exceed the following percent by mass of cement when tested in accordance with AASHTO T260:

Category	Acid-Soluble	Water-Soluble
Prestressed concrete	0.08	0.06
Reinforced concrete	0.10	0.08

Unless otherwise specified, the Contractor shall use Type I or II portland cement or blended hydraulic cement in all concrete as defined in Section 9-01.2(1).

The use of fly ash is required for Class 4000P concrete, except that ground granulated blast furnace slag may be substituted for fly ash at a 1:1 ratio. The use of fly ash and ground granulated blast furnace slag is optional for all other classes of concrete and may be substituted for portland cement at a 1:1 ratio as noted in the table below.

Cementitious Requirement for Concrete

Class of Concrete	Minimum Cementitious Content (Pounds)	Minimum percent Replacement of Fly Ash or Ground Granulated Blast Furnace Slag for Portland Cement	Maximum percent Replacement of Fly Ash for Portland Cement	Maximum percent Replacement of Ground Granulated Blast Furnace Slag for Portland Cement
4000	564	*	35	50
4000A	564	*	25	30
4000P	600	15	35	50
4000W	564	*	35	50
3000	564	*	35	50
Commercial Concrete	**564	*	35	50

*No minimum specified.

**For Commercial Concrete, the minimum cementitious content is only required for sidewalks, curbs, and gutters.

When both ground granulated blast furnace slag and fly ash are included in the concrete mix, the total weight of both these materials is limited to 40 percent by weight of the total cementitious material for concrete class 4000A, and 50 percent by weight of the total cementitious material for all other classes of concrete.

The water/cement ratio shall be calculated on the total weight of cementitious material. Cementitious materials are those listed in Section 5-05.2. With the Engineer’s written concurrence, microsilica fume may be used in all classifications of Class 4000, Class 3000, and commercial concrete and is limited to a maximum of 10 percent of the cementitious material.

As an alternative to the use of fly ash, ground granulated blast furnace slag and cement as separate components, a blended hydraulic cement that meets the requirements of Section 9-01.2(1)B Blended Hydraulic Cements may be used.

6-02.3(2)A Contractor Mix Design

The Contractor shall provide a mix design in writing to the Engineer for all classes of concrete specified in the Plans except for lean concrete Type 2, commercial concrete and concrete class EA. No concrete shall be placed until the Engineer has reviewed the mix design. The required average 28-day compressive strength shall be selected in accordance with ACI 301, Chapter 4, Section 4.2.3.3. ACI 211.1 shall be used to determine proportions. All proposed concrete mixes except Class 4000D shall meet the requirements in Cementitious Requirement for Concrete in Section 6-02.3(2).

The Contractor's submittal of a mix design shall be on WSDOT Form 350-040 and shall provide a unique identification for each mix design. A unique identification for the mix design is comprised of the combination of the Mix Design Number and the Concrete Plant Number. The mix design shall include the mix proportions per cubic yard, the proposed sources, the average 28-day compressive strength for which the mix is designed, the fineness modulus, and the water cement ratio. The mix design submittal shall also include test results no older than one year showing that the Aggregates do not contain Deleterious Substances in accordance with Section 9-03. Concrete placeability, workability, and strength shall be the responsibility of the Contractor. The Contractor shall notify the Engineer in writing of all mix design modifications.

Fine aggregate shall conform to Section 9-03.1(2) Class 1 or Class 2.

Coarse aggregate shall conform to Section 9-03. An alternate combined aggregate gradation conforming to Section 9-03.1(5) may also be used. The nominal maximum size aggregate for Class 4000P shall be ⅜ inch. The nominal maximum size aggregate for Class 4000A shall be 1 inch.

Nominal maximum size for concrete aggregate is defined as the smallest standard sieve opening through which the entire amount of the aggregate is permitted to pass.

A retarding admixture is required in concrete Class 4000P.

Air content for concrete Class 4000D shall conform to Section 6-02.3(2)A1. For all other concrete, air content shall be a minimum of 4.5 percent and a maximum of 7.5 percent for all concrete placed above the finished ground line unless noted otherwise.

6-02.3(2)A1 Contractor Mix Design for Concrete Class 4000D

All Class 4000D concrete shall conform to the following requirements:

1. Aggregate shall use combined gradation in accordance with Section 9-03.1(5) with a nominal maximum aggregate size of 1½ inches.
2. Permeability shall be less than 2,000 coulombs at 56 days in accordance with AASHTO T277.
3. Freeze-thaw durability shall be provided by one of the following methods:
    a. The concrete shall maintain an air content between 4.5 and 7.5 percent.
    b. The concrete shall maintain a minimum air content that achieves a durability factor of 90 percent, minimum, after 300 cycles in accordance with AASHTO T 161, Procedure A. This air content shall not be less than 3.0 percent. Test samples shall be obtained from concrete batches of a minimum of 3.0 cubic yards.
4. Shrinkage at 28 days shall be less than 0.032 percent in accordance with AASHTO T 160.
5. Density shall be measured in accordance with ASTM C138.

The Contractor shall submit the mix design in accordance with Section 6-02.3(2)A. The submittal shall include test reports for all tests listed above that follow the reporting requirements of the AASHTO/ASTM procedures. Mix designs using shrinkage reducing admixture shall state the specific quantity required. Samples for testing may be obtained from either laboratory or concrete plant batches. If concrete plant batches are used, the minimum batch size shall be 3.0 cubic yards. Testing samples of mixes using shrinkage reducing admixture shall use the admixture and full water amounts specified in the mix design submittal. The Contractor shall submit the mix design and all test reports to the Engineer at least 30 calendar days prior to the placement of concrete in the bridge deck.

6-02.3(2)A2 Contractor Mix Design for Self-Consolidating Concrete

Self-consolidating concrete (SCC) is concrete that is able to flow under its own weight and completely fill the formwork without the need for vibration while maintaining homogeneity, even in the presence of dense reinforcement. SCC shall be capable of being pumped, and of flowing through the steel reinforcing bar cage without segregation or buildup of differential head inside or outside of the steel reinforcing bar cage.

Type III cement may be used in SCC.

SCC may be used for the following concrete Structure elements:

1. All cast-in-place concrete elements except bridge decks, bridge approach slabs, and cast-in-place concrete elements excluded by the Special Provisions.
2. Prestressed concrete girders in accordance with Sections 6-02.3(25).
3. The following precast concrete elements:
    a. Precast roof, wall, and floor panels and retaining wall panels in accordance with Section 6-02.3(9).
    b. Precast reinforced concrete three-sided structures, box culverts and split box culverts in accordance with Section 6-20.
    c. Precast concrete barrier in accordance with Section 6-10.3(1).
    d. Precast concrete wall stem panels in accordance with Section 6-11.3(3).
    e. Precast concrete noise wall panels in accordance with Section 6-12.3(6).
    f. Structural earth wall precast facing panels in accordance with Section 6-13.3(4).
    g. Precast drainage structure elements in accordance with Section 9-05.50.
    h. Precast junction boxes, cable vaults, and pull boxes in accordance with Section 9-29.2.

The mix design submittal shall include items specified in Section 6-02.3(2)A and results of the following tests conducted on concrete that has slump flow within the slump flow range defined below:

1. Slump Flow.
    a. The mix design shall specify the target slump flow in inches, in accordance with WSDOT FOP for ASTM C1611. The slump flow range is defined as the target slump flow plus or minus 2-inches.
    b. The visual stability index (VSI) shall be less than or equal to 1, in accordance with ASTM C1611, Appendix X1, using Filling Procedure B.
    c. The T50 flow rate results shall be less than 6-seconds in accordance with ASTM C1611, Appendix X1, using Filling Procedure B.
2. Column Segregation.
    a. The maximum static segregation shall be 10-percent in accordance with ASTM C1610.
    b. The Maximum Hardened Visual Stability Index (HVSI) shall be 1 in accordance with AASHTO PP 58.
3. J ring test results for passing ability shall be less than or equal to 1.5-inches in accordance with the WSDOT FOP for ASTM C1621.

4. Rapid assessment of static segregation resistance of self-consolidating concrete using penetration test in accordance with ASTM C1712 shall be less than or equal to 15 mm.
5. Air content shall be tested in accordance with WSDOT Test Method T 818, and shall conform to Section 6-02.3(2)A.
6. Concrete unit weight results in pounds per cubic foot shall be recorded in accordance with AASHTO T 121, except that the concrete shall not be consolidated in the test mold.
7. The temperature of all concrete laboratory test samples shall be tested in accordance with AASHTO T 309 and shall conform to the placement limits specified in Section 6-02.3(4)D.

In lieu of a Contractor-Provided mix design for SCC for precast concrete barrier, precast drainage structures, and precast junction boxes, cable vaults, and pull boxes, a representative full-size example Structure element shall be cast for inspection by the Contracting Agency. The Contractor shall have the structure sawn in half for examination by the Contracting Agency to determine that segregation has not occurred. The Contracting Agency's acceptance of the sawn structure will constitute acceptance of the manufacturing facility's use of SCC, and a concrete mix design submittal will not be required. Precast units cast at a manufacturing facility shall provide this sample as a component of the precast fabricating facility's annual plant approval process. Precast units cast on site shall provide this sample prior to casting the additional units.

6-02.3(2)B Commercial Concrete

Commercial concrete shall have a minimum compressive strength at 28 days of 3,000 psi in accordance with AASHTO T 22. Commercial concrete placed above the finished ground line shall be air entrained and have an air content from 4.5 percent to 7.5 percent in accordance with FOP for AASHTO T 152. Commercial concrete does not require mix design or source approvals for cement, aggregate, and other admixtures.

Where concrete Class 3000 is specified for items such as, culvert headwalls, plugging culverts, concrete pipe collars, pipe anchors, monument cases, Type PPB, PS, I, FB and RM signal standards, pedestals, cabinet bases, guardrail anchors, fence post footings, sidewalks, concrete curbs, curbs and gutters, and gutters, the Contractor may use commercial concrete. If commercial concrete is used for sidewalks, concrete curbs, curbs and gutters, and gutters, it shall have a minimum cementitious material content of 564 pounds per cubic yard of concrete, shall be air entrained, and the tolerances of Section 6-02.3(5)C shall apply.

6-02.3(2)C Concrete Class EA

Concrete for members and surfaces specified to receive an exposed aggregate finish shall be Class EA. Concrete Class EA shall conform to the following requirements:

28 day compressive strength	3,600 psi (minimum)
Cement	610 pounds per cubic yard
Fine Aggregate Class 1	880 pounds per cubic yard
Coarse Aggregate Grading No. 67	2,160 pounds per cubic yard
Water (maximum)	270 pounds per cubic yard
Water/Cement Ratio (maximum)	0.44

A Type A water reducing admixture conforming to Section 9-23.6 shall be used in accordance with Section 6-02.3(3). Air content shall conform to Section 6-02.3(2)A.

Mixing water shall be the minimum required for satisfactory placement and shall not exceed the specified amount.

Aggregate weights are based on a specific gravity of 2.67. Adjustments in the mix design will be made by the Engineer as necessary to correct for actual bulk specific gravity of the aggregates, moisture content of the aggregates, and to ensure proper consistency, workability, and correct cement content per cubic yard of concrete.

6-02.3(2)D Lean Concrete

Lean concrete Type 1 and lean concrete Type 2 are self-compacting, cementitious, flowable material requiring no subsequent vibration or tamping to achieve consolidation.

Unless otherwise provided, when the specifications refer to "lean concrete" without being designated as Type 1 or Type 2, it shall be understood as meaning either lean concrete Type 1 or lean concrete Type 2, subject to their respective restrictions on use (found in other sections).

Lean concrete Type 1 or Type 2 shall meet the following requirements:

Requirements for Lean Concrete		
	Lean Concrete Type 1 (See note 1)	**Lean Concrete Type 2**
Min. 28 Day Strength	100 psi	N/A
Max. 28 Day Strength	300 psi	N/A
Max. water/cementitious content	2.0	2.0
Air Content	Not specified	Not specified
Basis for Acceptance	Certification of Compliance 6-02.3(5)B	Certification of Compliance 6-02.3(5)B
Is Vibration Required	No	No
Is a mix design submittal required?	Yes	No
Consistency Requirement	Pumpable and flowable (approximate slump 6 to 10 inches)	Flowable (approximate slump 6 to 10 inches)
Batching Requirements	Meet 6-02.3(4)	Meet 6-02.3(4)
Minimum Cementitious Content (Pounds)	No minimum specified	145
Maximum Cementitious Content (pounds)	No maximum specified	200
Minimum percent Replacement of Fly Ash or Ground Granulated Blast Furnace Slag for Portland Cement	No minimum specified	No minimum specified
Maximum percent Replacement of Fly Ash for Portland Cement	No maximum specified	30
Maximum percent Replacement of Ground Granulated Blast Furnace Slag for Portland Cement	No maximum specified	50

Note 1: The Contractor shall submit a mix design for lean concrete Type 1 on WSDOT Form 350-040, as a Type 2 Working Drawing in accordance with Section 1-05.3, and shall provide a unique identification for the mix design. A unique identification for the mix design is composed of the combination of the Mix Design Number and the Concrete Plant Number. The mix design shall include the mix proportions per cubic yard, the proposed sources, the water cement ratio, and supporting strength test data. The mix design may be developed using, as a guideline, ACI 229R, Report on Controlled Low-Strength Materials, with the understanding that the requirements of Section 6-02.3(2)D take precedence over conflicts with ACI 229R. Additionally, the mix design submittal shall include at least 15 consecutive sets of 28-day compressive strength tests (using ASTM D4832), which demonstrate that the mean and standard deviation provide not less than an 80% probability that tests will be between the minimum and maximum strengths specified. No lean concrete Type 1 shall be placed until the Engineer has reviewed and commented on the mix design.

6-02.3(3) Admixtures

Concrete admixtures shall be added to the concrete mix at the time of batching the concrete or in accordance with the manufacturer's written procedure and as accepted by the Engineer. A copy of the manufacturer's written procedure shall be furnished to the Engineer prior to use of admixtures. Deviations from the manufacturer's written procedures shall be submitted as a Type 2 Working Drawing. Admixtures shall not be added to the concrete with the modified procedures until the Engineer has concurred in writing.

When the Contractor is proposing to use admixtures from different admixture manufacturers they shall provide evidence to the Engineer that the admixture will be compatible and not adversely affect the air void system of the hardened concrete. Test results complying with ASTM C457 shall be provided as the evidence to satisfy this requirement. Admixture combinations which have been previously tested and which are in compliance with ASTM C457 shall be listed in the Qualified Products List (QPL). Proposed combinations not found in the QPL shall meet this requirement.

Accelerating admixtures conforming to Sections 9-23.6(4) or 9-23.6(6) and used in accordance with the manufacturer's recommendations may be used in cast-in-place concrete, except as required here. Accelerating admixtures shall not be used in bridge decks, all concrete superstructures, crossbeams, columns, mass concrete, or new bridge approach slabs and expansion joints that are not part of a repair. Concrete placements with the least dimension greater than 6 feet shall be considered mass concrete. Concrete placement with the least dimension greater than 3 feet, but less than or equal than 6 feet, shall require the approval of the Engineer for the use of accelerating admixtures. Shafts shall not be considered mass concrete. Chloride based accelerating admixtures shall not be used.

Air entrained cement shall not be used to air entrain concrete.

6-02.3(4) Ready-Mix Concrete

All concrete, unless otherwise specified, shall be batched in a prequalified manual, semi-automatic, or automatic plant as described in Section 6-02.3(4)A. The Engineer is not responsible for delays due to problems in getting the plant certified.

6-02.3(4)A Qualification of Concrete Suppliers

Batch Plant Prequalification requires a certification by the National Ready Mix Concrete Association (NRMCA). Information concerning NRMCA certification may be obtained from the NRMCA at 900 Spring Street, Silver Springs, MD 20910 or online at www.nrmca.org. The NRMCA certification shall be valid for a 2-year period from the date of certificate. The following documentation shall be submitted to the Engineer; a copy of the current NRMCA Certificate of Conformance, the concrete mix design(s) (WSDOT Form 350-040), along with copies of the truck list, batch plant scale certification, admixture dispensing certification, and volumetric water batching devices (including water meters) verification.

For central-mixed concrete, the mixer shall be equipped with a timer that prevents the batch from discharging until the batch has been mixed for the prescribed mixing time. A mixing time of 1 minute will be required after all materials and water have been introduced into the drum. Shorter mixing time may be allowed if the mixer performance is tested in accordance with (AASHTO M157 Annex A1 Concrete Uniformity Requirements). Tests shall be conducted by an independent testing lab or by a commercial concrete producer's lab. If the tests are performed by a producer's lab, the Engineer or a representative will witness all testing.

For shrink-mixed concrete, the mixing time in the stationary mixer shall not be less than 30 seconds or until the ingredients have been thoroughly blended.

For transit-mixed or shrink-mixed concrete, the mixing time in the transit mixer shall be a minimum of 70 revolutions at the mixing speed designated by the manufacturer of the mixer. Following mixing, the concrete in the transit mixer may be agitated at the manufacturer's designated agitation speed. A maximum of 320 revolutions (total of mixing and agitation) will be permitted prior to discharge.

All transit-mixers shall be equipped with an operational revolution counter and a functional device for measurement of water added. All mixing drums shall be free of concrete buildup and the mixing blades shall meet the minimum Specifications of the drum manufacturer. A copy of the manufacturer's blade dimensions and configuration shall be on file at the concrete producer's office. A clearly visible metal data plate (or plates) attached to each mixer and agitator shall display: (1) the maximum concrete capacity of the drum or container for mixing and agitating, and (2) the rotation speed of the drum or blades for both the agitation and mixing speeds. Mixers and agitators shall always operate within the capacity and speed-of-rotation limits set by the manufacturer. Mixers, when fully loaded, shall keep the concrete uniformly mixed. All mixers and agitators shall be capable of discharging the concrete at a steady rate. Only those transit-mixers which meet the above requirements will be allowed to deliver concrete to a Contracting Agency project covered by these Specifications.

In transit-mixing, mixing shall begin within 30 seconds after the cement is added to the aggregates.

Central-mixed concrete, transported by truck mixer/agitator, shall not undergo more than 250 revolutions of the drum or blades before beginning discharging. To remain below this limit, the suppler may agitate the concrete intermittently within the prescribed time limit. When water or admixtures are added after the load is initially mixed, an additional 30 revolutions will be required at the recommended mixing speed.

For each project, at least biannually, or as required, the Plant Manager will examine mixers and agitators to check for buildup of hardened concrete or worn blades. If this examination reveals a problem, or if the Engineer wishes to test the quality of the concrete, slump tests may be performed with samples taken at approximately the ¼ and ¾ points as the batch is discharged. The maximum allowable slump difference shall be as follows:

> If the average of the two slump tests is < 4 inches, the difference shall be < 1 inch or
> if the average of the two slump tests is >4 inches, the difference shall be < 1½ inches.

If the slump difference exceeds these limits, the equipment shall not be used until the faulty condition is corrected. However, the equipment may continue in use if longer mixing times or smaller loads produce batches that pass the slump uniformity tests.

All concrete production facilities will be subject to verification inspections at the discretion of the Engineer. Verification inspections are a check for: current scale certifications; accuracy of water metering devices; accuracy of the batching process; and verification of coarse aggregate quality.

If the concrete producer fails to pass the verification inspection, the following actions will be taken:

1. For the first violation, a written warning will be provided.
2. For the second violation, the Engineer will give written notification and the Contracting Agency will assess a price reduction equal to 15 percent of the invoice cost of the concrete that is supplied from the time of the infraction until the deficient condition is corrected.
3. For the third violation, the concrete supplier is suspended from providing concrete until all such deficiencies causing the violation have been permanently corrected and the plant and equipment have been reinspected and meets all the prequalification requirements.

4. For the fourth violation, the concrete supplier shall be disqualified from supplying concrete for 1 year from the date of disqualification. At the end of the suspension period the concrete supplier may request that the facilities be inspected for prequalification.

6-02.3(4)B Jobsite Mixing

For small quantities of concrete, the Contractor may mix concrete on the job site provided the Contractor has requested in writing and received written permission from the Engineer. The Contractor's written request shall include a mix design, batching and mixing procedures, and a list of the equipment performing the job-site mixing. All job site mixed concrete shall be mixed in a mechanical mixer.

If the Engineer permits, hand mixing of concrete will be permitted for pipe collars, pipe plugs, fence posts, or other items receiving the concurrence of the Engineer, provided the hand mixing is done on a watertight platform in a way that distributes materials evenly throughout the mass. Mixing shall continue long enough to produce a uniform mixture. No hand mixed batch shall exceed ½ cubic yard.

Concrete mixed at the jobsite is never permitted for placement in water.

6-02.3(4)C Consistency

The maximum slump for concrete shall be:

1. 3½ inches for vibrated concrete placed in all bridge decks, bridge approach slabs, and flat slab bridge Superstructures.
2. 4½ inches for all other vibrated concrete.
3. 7 inches for non-vibrated concrete. (Includes Class 4000P)
4. 9 inches for shafts when using Class 4000P, provided the water cement ratio does not exceed 0.44 and a water reducer is used meeting the requirements of Section 9-23.6.
5. 5½ inches for all concrete placed in curbs, gutters, and sidewalks.

When a high range water reducer is used, the maximum slump listed in 1, 2, 3, and 5 above, may be increased an additional 2 inches.

For self-consolidating concrete (SCC), the slump requirements specified above do not apply, and are instead replaced by the target slump flow and slump flow range specified as part of the SCC mix design.

6-02.3(4)D Temperature and Time For Placement

The maximum allowed time to discharge for all concrete is the time from when the cement is added to the concrete mixture until it is discharged from the transit vehicle or placed in the forms at a precasting facility.

Concrete used in precast items, regardless of concrete class, shall remain between 50°F and 90°F and be discharged within 1.5 hours of mixing. All other concrete shall be placed within the following temperature limits and the specified discharge times.

Concrete Class	Concrete Temperature (Fahrenheit)	Maximum Discharge Time (Hours)
4000D	55° min. to 75°	1.75
	76° to 80° max.	1.5
All Other Classes of Structural Concrete and Self-Consolidating Concrete	55° min. to 75°	1.75
	76° to 90° max.	1.5
Commercial Concrete, Lean Concrete, and Class EA	55° min. to 90° max.	1.75

When conditions are such that the concrete may experience an accelerated initial set, the Engineer may require a shorter time to discharge. The time to discharge in the above table may be extended 15 minutes upon request from the Contractor and concurrence of the Engineer. Time extensions greater than 15 minutes require a Type 3 Working Drawing submittal. The submittal shall include:

1. An explanation of why an extended placement time is necessary for the Work.
2. The proposed concrete mix design, including the specified dosage of chemical admixtures for the anticipated range of concrete temperatures and details regarding when the admixtures are to be introduced into the mix. Type B (retarding) or Type D (water-reducing and retarding) chemical admixtures are required for structural or self-consolidating concrete.
3. Technical data sheets and supporting information from the admixture supplier indicating the appropriate chemical admixture dosage for the anticipated concrete temperatures, haul times, and working times.
4. The haul distance and estimated range of haul times.
5. The estimated number of drum rotations during haul. Rotations shall not exceed 400.
6. The proposed maximum time to discharge for the mix(es) shall not exceed 3 hours.

6-02.3(5) Acceptance of Concrete

6-02.3(5)A General

Concrete for the following applications will be accepted based on a Certificate of Compliance to be provided by the supplier as described in Section 6-02.3(5)B:

1. Lean concrete.
2. Commercial concrete.
3. Class 4000P concrete for Roadside Steel Sign Support Foundations.
4. Class 4000P concrete for Type II, III, and CCTV Signal Standard Foundations that are 12’-0” or less in depth.
5. Class 4000P concrete for Type IV and V Strain Pole Foundations that are 12’-0” or less in depth.
6. Class 4000P concrete for Steel Light Standard Foundations Types A & B.

Concrete Class EA will be accepted based on conformance to the requirements specified in Section 6-02.3(2)C for proportioning, temperature, and 28 day compressive strength.

Slip-form barrier concrete will be accepted based on conformance to the requirements for temperature, air content and compressive strength at 28 days for sublots as tested and determined by the Contracting Agency.

All other concrete not listed herein will be accepted based on conformance to the requirement for temperature, slump, air content for concrete placed above finished ground line, and the specified compressive strength at 28 days for sublots as tested and determined by the Contracting Agency.

A sublot is defined as the material represented by an individual strength test. An individual strength test is the average compressive strength of cylinders from the same sample of material.

Each sublot will be deemed to have met the specified compressive strength requirement when both of the following conditions are met:

1. Individual strength tests do not fall below the specified strength by more than 12½ percent or 500 psi, whichever is least.
2. An individual strength test averaged with the two preceding individual strength tests meets or exceeds specified strength (for the same class and exact mix I.D. of concrete on the same Contract).

When compressive strengths fail to satisfy one or both of the above requirements, the Contractor may:

1. Request acceptance based on the Contractor/Suppliers strength test data for cylinders made from the same truckload of concrete as the Contracting Agency cylinders; provided:
    a. The Contractor's test results are obtained from testing cylinders fabricated, handled, and stored for 28 days in accordance with FOP for AASHTO R 100 and tested in accordance with AASHTO T 22. The test cylinders shall be the same size cylinders as those cast by the Contracting Agency.
    b. The technician fabricating the cylinders is qualified by either ACI, Grade 1 or WAQTC to perform this Work.
    c. The Laboratory performing the tests in accordance with AASHTO T 22 has an equipment calibration/certification system, and a technician training and evaluation process in accordance with AASHTO R-18.
    d. Both the Contractor and Contracting Agency have at least 15 test results from the same mix to compare. The Contractor's results could be used if the Contractor's computed average of all their test results is within one standard deviation of the Contracting Agency's average test result. The computed standard deviation of the Contractor's results must also be within plus or minus 200 psi of the Contracting Agency's standard deviation.
2. Request acceptance of in-place concrete strength based on core results. This method will not be used if the Engineer determines coring would be harmful to the integrity of the Structure. Cores, if allowed, will be obtained by the Contractor in accordance with AASHTO T 24 and delivered to the Contracting Agency for testing in accordance with AASHTO T 22. If the concrete in the Structure will be dry under service conditions, the core will be air dried at a temperature of between 60°F and 80°F and at a relative humidity of less than 60 percent for 7 days before testing, and will be tested air dry.

Acceptance for each sublot by the core method requires that the average compressive strength of three cores be at least 85 percent of the specified strength with no one core less than 75 percent of the specified strength. When the Contractor requests strength analysis by coring, the results obtained will be accepted by both parties as conclusive and supersede all other strength data for the concrete sublot.

If the Contractor elects to core, cores shall be obtained no later than 50 days after initial concrete placement. The Engineer will concur in the locations to be cored. Repair of cored areas shall be the responsibility of the Contractor. The cost incurred in coring and testing these cores, including repair of core locations, shall be borne by the Contractor.

6-02.3(5)B Certification of Compliance

The concrete producer shall provide a Certificate of Compliance for each truckload of concrete. The Certificate of Compliance shall verify that the delivered concrete is in compliance with the mix design and shall include:

Manufacturer plant (batching facility)
Contracting Agency Contract number
Date
Time batched
Truck No.
Initial revolution counter reading
Quantity (quantity batched this load)
Type of concrete by class and producer design mix number

Cement producer, type, and Mill Certification No. (The mill test number as required by Section 9-01.3 is the basis for acceptance of cement.)

Fly ash (if used) brand and Class
Accepted aggregate gradation designation

Mix design weight per cubic yard and actual batched weights for:

Cement
Fly ash (if used)
Coarse concrete aggregate and moisture content (each size)
Fine concrete aggregate and moisture content
Water (including free moisture in aggregates)
Admixtures brand and total quantity batched

Air-entraining admixture
Water-reducing admixture
Other admixture

For concretes that use combined aggregate gradation, the Certificate of Compliance shall include the aggregate components and moisture contents for each size in lieu of the aggregate information described above.

For commercial concrete, the Certificate of Compliance shall include, as a minimum, the batching facility, date, and quantity batched per load.

6-02.3(5)C Conformance to Mix Design

Cement, coarse and fine aggregate weights shall be within the following tolerances of the mix design:

Batch Volumes less than or equal to 4 cubic yards		
Cement	+5%	-1%
Aggregate	+10%	-2%

Batch Volumes more than 4 cubic yards		
Cement	+5%	-1%
Aggregate	+2%	-2%

If the total cementitious material weight is made up of different components, these component weights shall be within the following tolerances:

1. Cement weight plus 5 percent or minus 1 percent of that specified in the mix design.
2. Fly ash and ground granulated blast furnace slag weight plus or minus 5 percent of that specified in the mix design.
3. Microsilica weight plus or minus 10 percent of that specified in the mix design.

Water shall not exceed the maximum water specified in the mix design.

6-02.3(5)D Test Methods

Acceptance testing will be performed by the Contracting Agency in accordance with the WSDOT *Materials Manual* M 46-01. The test methods to be used with this Specification are:

AASHTO T 22	Compressive Strength of Cylindrical Concrete Specimens
FOP for AASHTO R 100	Making and Curing Concrete Test Specimens in the Field
FOP for AASHTO T 119	Slump of Hydraulic Cement Concrete
FOP for WAQTC TM 2	Sampling Freshly Mixed Concrete
FOP for AASHTO T 152	Air Content of Freshly Mixed Concrete by the Pressure Method

FOP for AASHTO T 231	Capping Cylindrical Concrete Specimens
FOP for AASHTO T 309	Temperature of Freshly Mixed Portland Cement Concrete
ASTM C1611	Standard Test Method for Slump Flow of Self-Consolidating Concrete (Inverted Mold Method only)
ASTM C1621	Standard Test Method for Passing Ability of Self-Consolidating Concrete by J-Ring (Inverted Mold Method only)

6-02.3(5)E Point of Acceptance

Determination of concrete properties for acceptance will be made based on samples taken as follows:

Bridge decks, overlays, bridge approach slabs, and barriers at the discharge of the placement system. All other placements at the truck discharge.

It shall be the Contractor's responsibility to provide adequate and representative samples of the fresh concrete to a location designated by the Engineer for the testing of concrete properties and making of cylinder specimens. Samples shall be provided as directed in Sections 1-06.1 and 1-06.2. Once the Contractor has turned over the concrete for acceptance testing, no more mix adjustment will be allowed. The concrete will either be accepted or rejected.

6-02.3(5)F Water/Cement Ratio Conformance

The actual water cement ratio shall be determined from the certified proportions of the mix, adjusting for on the job additions. No water may be added after acceptance testing or after placement has begun, except for concrete used in slip forming. For slip-formed concrete, water may be added during placement but shall not exceed the maximum water cement ratio in the mix design, and shall meet the requirements for consistency as described in Section 6-02.3(4)C. If water is added, an air and temperature test shall be taken prior to resuming placement to ensure that Specification conformance has been maintained.

6-02.3(5)G Sampling and Testing for Temperature, Consistency, and Air Content

Concrete properties shall be determined from concrete as delivered to the project and as accepted by the Contractor for placement. The Contracting Agency will perform acceptance testing on all concrete for temperature and air content, if applicable. Concrete that is not self-consolidating concrete will be tested for slump. The following additional acceptance tests will be performed on self-consolidating concrete:

1. Slump flow within the target slump flow range.
2. J ring passing ability less than or equal to 1.5 inches.
3. VSI less than or equal to 1.

Sampling and testing will be performed before concrete placement from the first load and then randomly performed from one load for every 100 cubic yards. Concrete shall not be placed until all tests have been completed by the Engineer, and the results indicate that the concrete is within acceptable limits. If at any time the concrete is not within acceptable limits, sampling and testing will continue before concrete placement for each load until two successive loads meet all of the applicable acceptance requirements. After two successive tests indicate that the concrete is within specified limits, the testing frequency may decrease to one for every 100 cubic yards. Sampling shall be performed in accordance with FOP for WAQTC TM 2 and random samples shall be selected in accordance with WSDOT T 716. After the first acceptable load of concrete, up to ½ cubic yard may be placed from subsequent loads to be tested prior to testing for acceptance.

When the results for all subsequent acceptance tests indicate that the concrete as delivered and approved by the Contractor for placement does not conform to the specified limits, the sampling and testing frequency will be resumed for each load. Whenever one subsequent test indicates that the concrete is within the specified limits, the random sampling and testing frequency of one for every 100 cubic yards may resume.

Sampling and testing for a placement of one class of concrete consisting of 50 cubic yards or less will be as listed above, except that after one set of tests indicate that the concrete is within specified limits, the remaining concrete to be placed may be accepted by visual inspection.

6-02.3(5)H Sampling and Testing for Compressive Strength and Initial Curing

Acceptance testing for compressive strength shall be conducted at the same frequency as the acceptance tests for temperature, consistency, and air content.

The Contractor shall provide and maintain a sufficient number of cure boxes in accordance with FOP for AASHTO R 100 for curing concrete cylinders. The cure boxes shall be readily accessible and no more than 500 feet from the point of acceptance testing, unless otherwise allowed by the Engineer. The Contractor shall also provide, maintain and operate all necessary power sources and connections needed to operate the cure boxes. The cure boxes shall be in-place and functioning at the specified temperature for curing cylinders prior to concrete placement. Concrete cylinders shall be cured in the cure boxes in accordance with FOP for AASHTO R 100. The cure boxes shall have working locks and the Contractor shall provide the Engineer with one key to each of the locks. Once concrete cylinders are placed in the cure box, the cure box shall not be disturbed until the cylinders have been removed. The Contractor shall retain the cure box Temperature Measuring Device log and provide it to the Engineer upon request.

The Contractor shall protect concrete cylinders in cure boxes from excessive vibration and shock waves during the curing period in accordance with Section 6-02.3(6)D.

All cure box costs shall be incidental to the associated item of work.

6-02.3(5)I Test Section for Cast-In-Place SCC

Unless otherwise approved by the Engineer, the Contractor shall construct a test section of the element being constructed of cast-in-place SCC. The Contractor shall confirm, through the SCC placement operation in the test section, the SCC flows the distance required, completely filling the forms and encapsulating the reinforcement as required without leaving voids and pockets and causing segregation of the SCC mix. The test section forms, reinforcing steel and concrete placing operations shall be identical to those to be used in the production elements.

For horizontal elements, the test section shall simulate the flow of concrete for the maximum distance anticipated during production concrete placement. The depth and width of the test section for horizontal element may be smaller than the actual depth and width of the element to be cast. For vertical elements, the test section shall be a minimum of 33-percent of the height of the tallest element to be constructed. The Contractor shall submit Type 2 Working Drawings consisting of formwork and reinforcement details of the test section and SCC placement procedures.

After removing the forms, the test section will be inspected for signs of honeycombs, cracks, aggregate segregation, sedimentation, cold joints, and other surface and concrete placement defects. If such defects are present, the Contractor shall revise the formwork and SCC placement procedures as necessary to eliminate such defects.

Acceptance of the test section and the SCC mix design is contingent on acceptable visual inspection, and a minimum of two 4-inch minimum diameter core samples taken from the placement location and the furthest-most limits of the concrete as identified by the Engineer. The number of core locations will be specified by the Engineer. The difference in average unit weight of the locations represented by the core samples shall be less than 5-percent.

The Contractor shall use the same SCC placement procedures confirmed by the Engineer accepted test section for casting the production members.

6-02.3(5)J SCC in Precast Units

SCC for concrete barrier will be accepted in accordance with temperature, air, and compressive strength testing listed in Section 6-02.3(9).

SCC for precast junction boxes, cable vaults, and pull boxes will be accepted in accordance with the temperature and compressive strength testing listed in Section 6-02.3(9).

SCC for precast drainage structure elements will be accepted in accordance with the requirements of AASHTO M199.

6-02.3(5)K Rejecting Concrete

Rejection Without Testing – The Engineer, prior to sampling, may reject any batch or load of concrete that appears defective in composition; such as cement content or aggregate proportions. Rejected material shall not be incorporated in the Structure.

6-02.3(5)L Concrete With Non-Conforming Strength

Concrete with cylinder compressive strengths (fc) that fail to meet acceptance level requirements shall be evaluated for structural adequacy. If the material is found to be adequate, payment shall be adjusted in accordance with the following formula:

$$\text{Pay adjustment} = \frac{2(f'c - fc)(UP)(Q)}{f'c}$$

Where:

f'c	=	Specified minimum compressive strength at 28 days.
fc	=	Compressive strength at 28 days as determined by AASHTO Test Methods.
UP	=	Unit Contract price per cubic yard for the class of concrete involved.
Q	=	Quantity of concrete represented by an acceptance test based on the required frequency of testing.

Concrete that fails to meet minimum acceptance levels using the coring method will be evaluated for structural adequacy. If the material is found to be adequate, payment shall be adjusted in accordance with the following formula:

$$\text{Pay adjustment} = \frac{3.56(.85f'c - f\ cores)(UP)(Q)}{f'c}$$

Where:

f'c	=	Specified minimum compressive strength at 28 days.
f cores	=	Compressive strength of the cores as determined by AASHTO T 22.
UP	=	Unit Contract price per cubic yard for the class of concrete involved.
Q	=	Quantity of concrete represented by an acceptance test based on the required frequency of testing.

Where these Specifications designate payment for the concrete on other than a per cubic yard basis, the unit Contract price of concrete shall be taken as $300 per cubic yard for concrete Class 4000, 5000, and 6000. For concrete Class 3000, the unit contract price for Concrete shall be $160 per cubic yard.

6-02.3(6) Placing Concrete

The Contractor shall not place concrete:

1. On frozen or ice-coated ground or Subgrade;
2. Against or on ice-coated forms, reinforcing steel, structural steel, conduits, precast members, or construction joints;

3. Under rainy conditions; placing of concrete shall be stopped before the quantity of surface water is sufficient to affect or damage surface mortar quality or cause a flow or wash the concrete surface;
4. In a foundation until the Engineer has accepted its depth and characteristics;
5. Until the Engineer has accepted the form and the placement of installed reinforcing; or
6. In a Work area when vibrations from nearby Work may harm the concrete's initial set or strength.

When a foundation excavation contains water, the Contractor shall pump it dry before placing concrete. If this is impossible, an underwater concrete seal shall be placed that complies with Section 6-02.3(6)B. This seal shall be thick enough to resist uplift.

All foundations, forms, and contacting concrete surfaces shall be moistened with water just before the concrete is placed. Standing water on the foundation, concrete surface, or forms shall be removed.

The Contractor shall place concrete in the forms as soon as possible after mixing. The concrete shall always be plastic and workable. For this reason, the Engineer may reduce the time to discharge even further. Concrete placement shall be continuous, with no interruption longer than 30 minutes between adjoining layers unless the Engineer allows a longer time. The Type 2 Working Drawing submittal shall include justification that the concrete mix design will remain fluid for interruptions longer than 30 minutes between placements. Each layer shall be placed and consolidated before the preceding layer takes initial set. After initial set, the forms shall not be jarred, and projecting ends of reinforcing bars shall not be disturbed.

In girders or walls, concrete shall be placed in continuous, horizontal layers 1½ to 2½ feet deep. Compaction shall leave no line of separation between layers. In each part of a form, the concrete shall be deposited as near its final position as possible.

Methods for placing and consolidating concrete shall not segregate aggregates or displace reinforcing steel. Methods shall leave a compact, dense, and impervious concrete with smooth faces on exposed surfaces. Plastering is not permitted. Defective concrete shall be removed at the Contractor's expense.

To prevent aggregates from separating, the conveyor belt used to transport concrete shall not exceed 300 feet in length. If the mix needs protection from sun or rain, the Contractor shall cover the belt. When concrete pumps are used for placement, a Contractor's representative shall, prior to use on the first placement of each day, visually inspect the pumps water chamber for water leakage. No pump shall be used that allows free water to flow past the piston.

If a concrete pump is used as the placing system, the pump priming slurry shall be discarded before placement. Initial acceptance testing may be delayed until the pump priming slurry has been eliminated from the concrete being pumped. Eliminating the priming slurry from the concrete may require that several cubic yards of concrete are discharged through the pumping system and discarded. Use of a concrete pump requires a reserve pump (or other backup equipment) at the site.

If the concrete will drop more than 5 feet, it shall be deposited through a sheet metal (or other accepted) conduit. If the form slopes, the concrete shall be lowered through accepted conduit to keep it from sliding down one side of the form. No aluminum conduits or tremies shall be used to pump or place concrete. If aluminum concrete truck end chutes are used, concrete shall be continuously discharged in a manner that minimizes contact time between the concrete and the chute.

Before placing bridge deck concrete on steel spans, the Contractor shall release the falsework under the bridge and let the span swing free on its supports. Concrete in flat slab bridges shall be placed in one continuous operation for each span or series of continuous spans.

Concrete for bridge decks and the stems of T-beams or box-girders shall be placed in separate operations if the stem of the beam or girder is more than 3 feet deep. First the beam or girder stem shall be filled to the bottom of the slab fillets. Bridge deck concrete shall not be placed until enough time has passed to permit the earlier concrete to shrink (at least 12 hours). If stem depth is 3 feet or less, the Contractor may place concrete in 1 continuous operation if the Engineer concurs.

Between expansion or construction joints, concrete in beams, girders, bridge decks, piers, columns, walls, and traffic and pedestrian barriers shall be placed in a continuous operation.

After the concrete has been consolidated and prior to the application of cure, all surfaces of concrete that are not placed against forms shall be struck off to the planned elevation or slope and the surface shall be finished by floating with a float to seal the surface.

No traffic or pedestrian barrier shall be placed until after the bridge deck is complete for the entire Structure. No concrete barriers shall be placed until the falsework has been released and the span supports itself. The Contractor may choose not to release the deck overhang falsework prior to the barrier placement. The Contractor shall submit a Type 2E Working Drawing consisting of calculations indicating the loads induced into the girder webs due to the barrier weight and all live loads placed on the Structure do not exceed the design capacity of the girder component. This analysis is not required for bridges with concrete Superstructures. No barrier, curb, or sidewalk shall be placed on steel or prestressed concrete girder bridges until the bridge deck reaches a compressive strength of at least 3,000 psi.

The Contractor may construct traffic and pedestrian barriers by the slipform method. However, the barrier may not deviate more than ¼ inch when measured by a 10-foot straightedge held longitudinally on the front face, back face, and top surface. Electrical conduit within the barrier shall be constructed in accordance with the requirements of Section 8-20.3(5).

When placing concrete in arch rings, the Contractor shall ensure that the load on the falsework remains symmetrical and uniform.

Unless otherwise allowed by the Engineer, arch ribs in open spandrel arches shall be placed in sections. Small key sections between large sections shall be filled after the large sections have shrunk.

6-02.3(6)A Weather and Temperature Limits to Protect Concrete

6-02.3(6)A1 Hot Weather Protection

The Contractor shall provide concrete within the specified temperature limits. Cooling of the coarse aggregate piles by sprinkling with water is permitted provided the moisture content is monitored, the mixing water is adjusted for the free water in the aggregate and the coarse aggregate is removed from at least 1 foot above the bottom of the pile. Sprinkling of fine aggregate piles with water is not allowed. Refrigerating mixing water, or replacing all or part of the mixing water with crushed ice is permitted, provided the ice is completely melted by placing time.

If air temperature exceeds 90°F, the Contractor shall use water spray or other accepted methods to cool all concrete-contact surfaces to less than 90°F. These surfaces include forms, reinforcing steel, steel beam flanges, and any others that touch the concrete.

6-02.3(6)A2 Cold Weather Protection

Concrete shall be maintained at or above a temperature of 40°F during the first seven days of the Cold Weather Protection Period and at or above a temperature of 35°F during the remainder of the Cold Weather Protection Period. Cold weather protection requirements do not apply to concrete in shafts and piles placed below the ground line.

Prior to placing concrete in cold weather, the Contractor shall submit a Type 2 Working Drawing with a written procedure for cold weather concreting. The procedure shall detail how the Contractor will adequately cure the concrete and prevent the concrete temperature from falling below the minimum temperature. Extra protection shall be provided for areas especially vulnerable to freezing (such as exposed top surfaces, corners and edges, thin sections, and concrete placed into steel forms). Concrete placement will only be allowed if the Contractor's cold weather protection plan has been accepted by the Engineer.

Prior to concrete placement, the Contractor shall review the 7-day temperature predictions for the job site from the Western Region Headquarters of the National Weather Service (www.wrh.noaa.gov). When temperatures below 35°F are predicted, the Contractor shall:

1. Install temperature sensors in each concrete placement. One sensor shall be installed for every 100 cubic yards of concrete placed. Sensors shall be installed at locations directed by the Engineer, and shall be placed 1.5 inches from the face of concrete.
2. Immediately after concrete placement, temperature sensors shall be installed on the concrete surface at locations directed by the Engineer. One sensor shall be installed for every 100 cubic yards of concrete placed.

Temperatures shall be measured and recorded a minimum of every hour for the duration of the Cold Weather Protection Period. Temperature data shall be submitted to the Engineer as a Type 1 Working Drawing within three days following the end of the Cold Weather Protection Period.

For each day that the concrete temperature falls below 40°F during the first seven days of the Cold Weather Protection Period, no curing time is awarded for that day and the Cold Weather Protection Period is extended for one additional day. If the concrete temperature falls below 35°F during the Cold Weather Protection Period, the concrete may be rejected by the Engineer.

6-02.3(6)B Placing Concrete in Foundation Seals

If the Plans require a concrete seal, the Contractor shall place the concrete underwater inside a watertight cofferdam, tube, or caisson. Seal concrete shall be placed in a compact mass in still water. It shall remain undisturbed and in still water until fully set. While seal concrete is being deposited, the water elevation inside and outside the cofferdam shall remain equal to prevent any flow through the seal in either direction. The cofferdam shall be vented at the vent elevation shown in the Plans. The thickness of the seal is based upon this vent elevation.

The seal shall be at least 18 inches thick unless the Plans show otherwise. The Engineer may change the seal thickness during construction which may require redesign of the footing and the pier shaft or column. Although seal thickness changes may result in the use of more or less concrete, reinforcing steel, and excavation, payment will remain as originally defined in unit Contract prices.

To place seal concrete underwater, the Contractor shall use a concrete pump or tremie. The tremie shall have a hopper at the top that empties into a watertight tube at least 10 inches in diameter. The discharge end of the tube on the tremie or concrete pump shall include a device to seal out water while the tube is first filled with concrete. Tube supports shall permit the discharge end to move freely across the entire Work area and to drop rapidly to slow or stop the flow. One tremie may be used to concrete an area up to 18 feet per side. Each additional area of this size requires one additional tremie.

Throughout the underwater concrete placement operation, the discharge end of the tube shall remain submerged in the concrete and the tube shall always contain enough concrete to prevent water from entering. The concrete placement shall be continuous until the Work is completed, resulting in a seamless, uniform seal. If the concreting operation is interrupted, the Engineer may require the Contractor to prove by core drilling or other tests that the seal contains no voids or horizontal joints. If testing reveals voids or joints, the Contractor shall repair them or replace the seal at no expense to the Contracting Agency.

Concrete Class 4000W shall be used for seals, and it shall meet the consistency requirements of Section 6-02.3(4)C.

6-02.3(6)C Dewatering Concrete Seals and Foundations

After a concrete seal is constructed, the Contractor shall pump the water out of the cofferdam and place the rest of the concrete in the dry. This pumping shall not begin until the seal has set enough to withstand the hydrostatic pressure (3 days for gravity seals and 10 days for seals containing piling or shafts). The Engineer may extend these waiting periods to ensure structural safety or to meet a condition of the operating permit.

If weighted cribs are used to resist hydrostatic pressure at the bottom of the seal, the Contractor shall anchor them to the foundation seal. Any method used (such as dowels or keys) shall transfer the entire weight of the crib to the seal.

No pumping shall be done during or for 24 hours after concrete placement unless done from a suitable sump separated from the concrete Work by a watertight wall. Pumping shall be done in a way that rules out any chance of concrete being carried away.

6-02.3(6)D Protection Against Vibration

Freshly placed concrete shall not be subjected to excessive vibration and shock waves during the curing period until it has reached a 2,000 psi minimum compressive strength for structural concrete and lower-strength classes of concrete.

After the first 5 hours from the time the concrete has been placed and consolidated, the Contractor shall keep all vibration producing operations at a safe horizontal distance from the freshly placed concrete by following either the prescriptive safe distance method or the monitoring safe distance method. These requirements for the protection of freshly placed concrete against vibration shall not apply for plant cast concrete, nor shall they apply to the vibrations caused by the traveling public.

6-02.3(6)D1 Prescriptive Safe Distance Method

After the concrete has been placed and consolidated, the Contractor shall keep all vibration producing operations at a safe horizontal distance from the freshly placed concrete as follows:

Minimum Compressive Strength, f’c	Safe Horizontal Distance[1]	
	Equipment Class L[2]	Equipment Class H[3]
< 1,000 psi	75 feet	125 feet
1,000 to < 1,400 psi	30 feet	50 feet
1,400 to 2,000 psi	15 feet	25 feet

[1]The safe horizontal distance shall be reduced to 10 feet for small rubber tire construction equipment like backhoes under 50,000 pounds, concrete placing equipment, and legal Highway vehicles if such equipment travels at speeds of:

- ≤ 5 mph on relatively smooth Roadway surfaces or
- ≤ 3 mph on rough Roadway surfaces (i.e., with potholes)

[2]Equipment Class L (Low Vibration) shall include tracked dozers under 85,000 pounds, track vehicles, trucks (unless excluded above), hand-operated jack hammers, cranes, auger drill rig, caisson drilling, vibratory roller compactors under 30,000 pounds, and grab-hammers.

[3]Equipment Class H (High Vibration) shall include pile drivers, vibratory hammers, machine-operated impact tools, pavement breakers, and other large pieces of equipment.

After the concrete has reached a minimum compressive strength specified above, the safe horizontal distance restrictions would no longer apply.

6-02.3(6)D2 Monitoring Safe Distance Method

The Contractor may monitor the vibration producing operations in order to decrease the safe horizontal distance requirements of the prescriptive safe distance method. If this method is chosen, all construction operations that produce vibration or shock waves in the vicinity of freshly placed concrete shall be monitored by the Contractor with monitoring equipment sensitive enough to detect a minimum peak particle velocity (PPV) of 0.10 inches per second. Monitoring devices shall be placed on or adjacent to the freshly placed concrete when the measurements are taken. During the time subsequent to the concrete placement, the Contractor shall cease all vibration or shock producing operations in the vicinity of the newly placed concrete when the monitoring equipment detects excessive vibration and shock waves defined as exceeding the following PPVs:

Minimum Compressive Strength, f’c	Maximum PPV
< 1,000 psi	0.10 in/sec
1,000 to < 1,400 psi	1.0 in/sec
1,400 to 2,000 psi	2.0 in/sec

After the concrete has reached a minimum compressive strength specified above, the safe horizontal distance restrictions would no longer apply.

6-02.3(7) Tolerances

Unless noted otherwise, concrete construction tolerances shall be in accordance with this section. Tolerances in this section do not apply to cement concrete pavement.

Horizontal deviation of roadway crown points, cross-slope break points, and curb, barrier or railing edges from alignment or work line: ±1.0 inch

Deviation from plane: ±0.5 inch in 10 feet

Deviation from plane for roadway surfaces: ±0.25 inch in 10 feet

Deviation from plumb or specified batter: ±0.5 inch in 10 feet, but not to exceed a total of ±1.5 inches

Plumbness or vertical deviation of webs for precast concrete girders and beams at points of support after erection: ±⅛ inch in 1 foot

Vertical deviation from profile grade for roadway surfaces: ±1 inch

Vertical deviation of top surfaces (except roadway surfaces): ±0.75 inch

Thickness of bridge decks and other structural slabs not at grade: ±0.25 inch

Length, width and thickness of elements such as columns, beams, crossbeams, diaphragms, corbels, piers, abutments and walls, including dimensions to construction joints in initial placements: +0.5 inch, -0.25 inch

Length, width and thickness of spread footing foundations: +2 inches, -0.5 inch

Horizontal location of the as-placed edge of spread footing foundations: The greater of ±2% of the horizontal dimension of the foundation perpendicular to the edge and ±0.5 inch. However, the tolerance shall not exceed ±2 inches.

Location of opening, insert or embedded item at concrete surface: ±0.5 inch

Cross-sectional dimensions of opening: ±0.5 inch

Bridge deck, bridge approach slab, and bridge traffic barrier expansion joint gaps with a specified temperature range, measured at a stable temperature: ±0.25 inch

Horizontal deviation of centerline of bearing pad, oak block or other bearing assembly: ±0.125 inch

Horizontal deviation of centerline of supported element from centerline of bearing pad, oak block or other bearing assembly ±0.25 inch

Vertical deviation of top of bearing pad, oak block or other bearing assembly: ±0.125 inch

6-02.3(8) Vibration of Concrete

The Contractor shall supply enough vibrators to consolidate the concrete (except that placed underwater) according to the requirements of this section. Each vibrator shall:

1. Be designed to operate while submerged in the concrete,
2. Vibrate at a rate of at least 7,000 pulses per minute, and
3. Receive the Engineer’s acceptance on its type and method of use.

Immediately after concrete is placed, vibration shall be applied in the fresh batch at the point of deposit. In doing so, the Contractor shall:

1. Space the vibrators evenly, no farther apart than twice the radius of the visible effects of the vibration;
2. Ensure that vibration intensity is great enough to visibly affect a weight of 1-inch slump concrete across a radius of at least 18 inches;
3. Insert the vibrators slowly to a depth that will effectively vibrate the full depth of each layer, penetrating into the previous layer on multilayer pours;
4. Protect partially hardened concrete (i.e., nonplastic, which prevents vibrator penetration when only its own weight is applied) by preventing the vibrator from penetrating it or making direct contact with steel that extends into it;
5. Not allow vibration to continue in one place long enough to form pools of grout;
6. Continue vibration long enough to consolidate the concrete thoroughly, but not so long as to segregate it;
7. Withdraw the vibrators slowly when the process is complete; and
8. Not use vibrators to move concrete from one point to another in the forms.

When vibrating and finishing top surfaces that will be exposed to weather or wear, the Contractor shall not draw water or laitance to the surface. In high lifts, the top layer shall be shallow and made up of a concrete mix as stiff as can be effectively vibrated and finished.

To produce a smooth, dense finish on outside surfaces, the Contractor shall hand tamp the concrete.

Vibration of SCC shall only be used as described below or as approved by the Engineer:

1. To prevent the formation of a cold joint in between placement of successive batches of SCC.
2. Near the end of an SCC placement to aid in leveling the SCC in the forms.

When vibration of SCC is allowed, the magnitude and duration of the applied vibration shall be kept as minimal as possible.

6-02.3(9) Precast Concrete Units

Precast concrete units may be cast at a manufacturing facility or cast on-site within the project limits unless otherwise stated in these Specifications. Pretensioned prestressed precast concrete units are prohibited from being cast on-site.

The manufacturing facility shall be certified by the Precast/Prestressed Concrete Institute's Plant Certification Program for the type of precast member to be produced, or the National Precast Concrete Association's Plant Certification Program or be an International Congress Building Officials or International Code Council Evaluation Services recognized fabricator of structural precast concrete products, and shall be approved by WSDOT as a Certified Precast Concrete Fabricator prior to the start of production. WSDOT Certification will be granted at, and renewed during, the annual precast plant review and approval process in accordance with WSDOT Materials Manual M 46-01 Standard Practice QC 7. Products that require annual plant approval include noise barrier panels, wall panels, floor and roof panels, marine pier deck panels, retaining walls, pier caps, and bridge deck panels. Precast concrete panels that are prestressed shall meet all the requirements of Section 6-02.3(25).

Precast units that are cast within the project limits shall be considered cast on-site and are exempt from the fabrication plant approval requirement for precast products. Cast on-site units shall meet all the same quality control standards as a manufacturing facility, and all the requirements in these Specifications. Additionally, the Contractor shall submit a Type 2E Working Drawing consisting of an On-site Pre-casting and Quality Control Plan for review prior to beginning any on-site precast work. This may be a Type 2 Working Drawing if item 6 does not apply. The On-site Pre-casting and Quality Control Plan shall include at a minimum the following items:

1. List of unit(s) to be cast on-site
2. Name of designated Quality Control Supervisor for all on-site casting operations
3. Location for on-site casting, curing, and storage
4. On-site casting quality control plan and procedures
5. Concrete mix design
6. Calculation of required compressive strength if unit is to be removed from the form prior to the concrete strength reaching 70% of the specified design strength, as specified in Section 6-02.3(9)B.

The Contractor shall be responsible for quality control inspection on all precast concrete units. Prior to the start of production of the precast concrete units, the Contractor shall advise the Engineer of the production schedule. The Contractor shall give the Inspector safe and free access to the Work. If the Inspector observes non-specification Work or unacceptable quality control practices, the Inspector will advise the plant manager if cast at a manufacturing facility or the Contactor's Quality Control Supervisor if cast on-site. If the corrective action is not acceptable to the Engineer, the units will be subject to rejection.

Type III portland cement or blended hydraulic cement is permitted to be used in precast concrete units.

Self-consolidating concrete (SCC) may be used in accordance with Section 6-02.3(2)A.

Acceptance testing shall be performed by the Contractor and test results shall be submitted to the Engineer when cast at a manufacturing facility. When cast on site, acceptance testing shall be performed by WSDOT. Concrete shall conform to the requirements specified in Section 6-02.3(2)A and Section 6-02.3(5), unless otherwise noted. The test methods described in Section 6-02.3(5)D shall be followed, unless otherwise noted. Compressive strength testing shall be performed a minimum of once per day and once for every 20 cubic yards of concrete that is placed.

6-02.3(9)A Shop Drawings

Before casting the structural elements, the Contractor shall submit Type 2 Working Drawings of the precast unit shop drawings.

These shop drawings shall show complete details of the methods, materials, and equipment the Contractor proposes to use in prestressing/precasting Work. The shop drawings shall follow the design conditions shown in the Plans and accepted On-site Pre-casting Quality Control Plan, if applicable, unless the Engineer concurs with equally effective variations.

The shop drawings shall contain as a minimum:

1. Unit shapes (elevations and sections) and dimensions.
2. Finishes and method of constructing the finish (i.e., forming, rolling).
3. Reinforcing, joint, and connection details.
4. Location and type of lifting, bracing, and erection inserts including manufacturer's recommended safe working capacity.
5. Material specifications
6. Locations and details of hardware attached to the Structure.
7. Relationship to adjacent material.

Deviations from the approved shop drawings shall only be permitted after submitting a Type 2 Working Drawing that describes the proposed changes.

Before completion of the Contract, the Contractor shall provide the Engineer with shop drawings (which include all processed changes). These shall be clear and in a format that conforms with Section 6-01.9.

6-02.3(9)B Casting

Before casting precast concrete units, the Contractor and Fabrication Inspector or Engineer shall have possession of a processed set of shop drawings.

Concrete shall meet the requirements of Section 6-02.3(25)C for annual preapproval of the concrete mix design and slump. Concrete for cast on-site units shall be in accordance with the accepted On-site Pre-casting and Quality Control Plan.

Precast units shall not be removed from forms until the concrete has attained a minimum compressive strength of 70 percent of the specified design strength. This is referred to as the stripping strength. A minimum compressive strength less than 70 percent may be used for the stripping strength for precast units if calculations are submitted and accepted in a Type 2E Working Drawing computed in accordance with the PCI Design Handbook indicating the required compressive strength for product handling. The calculations shall include, at a minimum, the effects of stripping, rigging configuration, stripping method, form suction, impact factors, effects of bunking, and the recommended safety factor for the modulus of rupture. Curing requirements of Section 6-02.3(9)C shall be maintained until 70% of the specified design strength is achieved.

Forms may be steel or plywood faced, providing they impart the required finish to the concrete.

6-02.3(9)C Curing

Concrete in the precast units shall be cured by either moist or accelerated curing methods. The curing methods to be used by a manufacturing facility shall be preapproved in the WSDOT plant certification process. The methods to be used when cast on-site shall be as accepted in the On-site Pre-casting and Quality Control Plan.

1. For moist curing, the surface of the concrete shall be kept covered or moist until such time as the compressive strength of the concrete reaches 70% of the specified design strength. Exposed surfaces shall be kept continually moist by fogging, spraying, or covering with moist burlap or cotton mats. Moist curing shall commence as soon as possible following completion of surface finishing.
2. For accelerated curing, heat shall be applied at a controlled rate following the initial set of concrete in combination with an effective method of supplying or retaining moisture. Moisture may be applied by a cover of moist burlap, cotton matting, or other effective means. Moisture may be retained by covering the unit with an impermeable sheet.

   Heat may be radiant, convection, conducted steam or hot air. Heat the concrete to no more than 100°F during the first 2 hours after placing the concrete, and then increase no more than 25°F per hour to a maximum of 175°F. After curing is complete, cool the concrete no more than 25°F per hour to 100°F. Maintain the concrete temperature above 60°F until the unit reaches stripping strength.
   Concrete temperature shall be monitored by means of a thermocouple embedded in the concrete (linked with a thermometer accurate to plus or minus 5°F). The recording sensor (accurate to plus or minus 5°F) shall be arranged and calibrated to continuously record, date, and identify concrete temperature throughout the heating cycle. This temperature record shall be made available to the Engineer for inspection and become a part of the documentation required.

   The Contractor shall never allow dry heat to directly touch exposed unit surfaces at any point.

6-02.3(9)D Control Strength

6-02.3(9)D1 Control Strength for Precast Units at a Fabrication Plant

Verification of the release strength and the design strength shall be determined by testing cylinders made from the same concrete as the precast units. The cylinders shall be made, handled, and stored in accordance with WSDOT FOP for AASHTO R 100 and compression tested in accordance with AASHTO T 22 and AASHTO T 231.

For accelerated cured units, concrete strength shall be measured on test cylinders cast from the same concrete as that in the unit. These cylinders shall be cured under time-temperature relationships and conditions that simulate those of the unit. If the forms are heated by steam or hot air, test cylinders will remain in the coolest zone throughout curing. If forms are heated another way, the Contractor shall provide a record of the curing time- temperature relationship for the cylinders for each unit to the Engineer. When two or more units are cast in a continuous line and in a continuous operation, a single set of test cylinders may represent all units provided the Contractor demonstrates uniformity of casting and curing to the satisfaction of the Engineer. The Contractor shall let cylinders cool for at least ½ hour before testing for stripping strength.

6-02.3(9)D2 Control Strength for Precast Units Cast On-Site

The Contractor shall sample and test concrete to verify that the stripping strength has been met. The Contractor shall cast enough cylinders to accomplish the verification. The Contractor shall be responsible for timing the testing of the cylinders and reporting the results to the Engineer. Sampling and testing shall be in accordance with the test methods specified in 6-02.3(5)D. To measure stripping strength in the precast unit, the Contractor shall randomly select two test cylinders and average their compressive strengths. The compressive strength in each cylinder shall not fall more than 5 percent below the specified strength. If these two cylinders do not pass the test, two other cylinders shall be selected and tested.

The Contracting Agency will sample and test the cylinders for purposes of verification of the design strength at 28-days.

6-02.3(9)E Finishing

The Contractor shall provide a finish on all relevant concrete surfaces as defined in Section 6-02.3(14), unless the Plans or Special Provisions require otherwise.

6-02.3(9)F Tolerances

The precast units shall be fabricated as shown in the Plans, and shall meet the dimensional tolerances listed in the latest edition of PCI-MNL-116, unless otherwise required by the Specifications, Plans or Special Provisions.

6-02.3(9)G Handling and Storage

The Contractor shall lift all units only by adequate devices at locations designated on the shop drawings.

The precast units shall not be stored or handled in a manner such that the stresses imposed on the structure, as presented in the On-site Pre-casting and Quality management Plan, are exceeded or cause damage to the structure including cracking and spalling. The Contactor shall submit lifting calculations as a supplement to the On-Site Pre-Casting Quality Control Plan at the request of the Engineer.

Precast units shall be stored off the ground on foundations suitable to prevent differential settlement or twisting of the unit. Stacked units shall be separated and supported by dunnage of uniform thickness capable of supporting the unit. Dunnage shall be arranged in vertical planes. The upper units of a stacked tier shall not be used as storage areas for shorter units unless submitted as a Type 2E Working Drawing containing engineering analysis and accepted by the Engineer.

Precast units with hairline cracks visibly apparent, radiating from the lifting loops or support locations extending more than three inches along the structure, or with hairline cracks in other locations, will be subject to evaluation by the Engineer for possible rejection. Precast units whose lifting loops pull out will be subject to evaluation by the Engineer for possible rejection.

6-02.3(9)H Shipping

Precast units shall not be shipped or transported if cast on-site until the concrete has reached the specified design strength, and the Engineer has reviewed the fabrication documentation for Contract compliance. Units cast at a manufacturing facility shall be stamped “Approved for Shipment”. Units cast on site shall not be transported to their permanent location until approved by the Engineer. The units shall be supported in such a manner that they will not be damaged by anticipated impact on their dead load. Sufficient padding material shall be provided between tie chains and cables to prevent chipping or spalling of the concrete.

6-02.3(9)I Erection

Precast units shall not be erected until the concrete has reached the specified design strength, and the Engineer has reviewed the fabrication documentation for Contract compliance. When the precast units arrive on the project from the manufacturing facility, the Engineer will confirm that they are stamped "Approved for Shipment". The Engineer will evaluate the present units for damage before accepting them. Units cast on-site shall be inspected and approved by the Engineer prior to erection.

The Contractor shall lift all precast units by suitable devices at locations designated on the shop drawings. Temporary shoring or bracing shall be provided, if necessary. Precast units shall be properly aligned and leveled as required by the Plans. Variations between adjacent elements shall be leveled out by a method accepted by the Engineer.

6-02.3(10) Bridge Decks and Bridge Approach Slabs

6-02.3(10)A Pre-Deck Pour Meeting

A pre-deck pour meeting shall be held 5 to 10 working days before placing deck concrete to discuss construction procedures, personnel, equipment to be used, concrete sampling and testing and deck finishing and curing operations. Those attending shall include, at a minimum, the superintendent, foremen in charge of placing and finishing concrete, and representatives from the concrete supplier and the concrete pump truck supplier.

If the project includes more than one bridge deck, and if the Contractor's key personnel change between concreting operations, or at request of the Engineer, additional conferences shall be held before each deck placement.

6-02.3(10)B Screed Rail Supports

The Contractor shall place screed rails outside the finishing area. When screed rails cannot be placed outside the finishing area as determined by the Engineer, they shall rest on adjustable supports that can be removed with the least possible disturbance to the screeded concrete. The supports shall rest on structural members or on forms rigid enough to resist deflection. Supports shall be removable to at least 2 inches below the finished surface. For staged constructed bridge decks, the finishing machine screed rails shall not be supported on the completed portion of deck and shall deflect with the portion of structure under construction.

Screed rails (with their supports) shall be strong enough and stiff enough to permit the finishing machine to operate effectively on them. All screed rails shall be placed and secured for the full length of the deck/slab before the concreting begins. If the Engineer concurs in advance, the Contractor may move rails ahead onto previously set supports while concreting progresses. However, such movable rails and their supports shall not change the set elevation of the screed.

On steel truss and girder spans, screed rails and bulkheads may be placed directly on transverse steel floorbeams, with the strike-board moving at right angles to the centerline of the Roadway.

6-02.3(10)C Finishing Equipment

The finishing machine shall be self-propelled and be capable of forward and reverse movement under positive control. The finishing machine shall be equipped with augers and a rotating cylindrical single or double drum screed. The finishing machine shall have the necessary adjustments to produce the required cross section, line, and grade. The finishing machine shall be capable of raising the screeds, augers, and any other parts of the finishing mechanical operation to clear the screeded surface and returning to the specified grade under positive control. Unless otherwise allowed by the Engineer, a finishing machine manufacturer technical representative shall be on site to assist the first use of the machine on the Contract.

For bridge deck widening of 20 feet or less, and for bridge approach slabs, or where jobsite conditions do not allow the use of the conventional configuration finishing machines, or modified conventional machines as described above, the Contractor may submit a Type 2 Working Drawing proposing the use of a hand-operated motorized power screed such as a “Texas” or “Bunyan” screed. This screed shall be capable of finishing the bridge deck and bridge approach slab to the same standards as the finishing machine.

On bridge decks, the Contractor may use hand-operated strike-boards only when the Engineer concurs for special conditions where self-propelled or motorized hand-operated screeds cannot be employed. These boards shall be sturdy and able to strike off the full placement width without intermediate supports. Strike-boards, screed rails, and all other specially made auxiliary equipment shall receive the Engineer’s concurrence before use. All finishing requirements in these Specifications apply to hand-operated finishing equipment.

6-02.3(10)D Concrete Placement, Finishing, and Texturing

6-02.3(10)D1 Test Slab Using Bridge Deck Concrete

After the Contractor receives the Engineer’s acceptance of the Class 4000D concrete mix design, and a minimum of seven calendar days prior to the first placement of bridge deck concrete, the Contractor shall construct a test slab using concrete of the accepted mix design.

The test slab may be constructed on grade, shall have a minimum thickness of 8-inches, shall have minimum plan dimensions of 10-feet along all four edges, and shall be square or rectangular.

During construction of the test slab, the Contractor shall demonstrate concrete sampling and testing, use of the concrete temperature monitoring system, the concrete fogging system, concrete placement system, and the concrete finishing operation. The Contractor shall conduct the demonstration using the same type of equipment to be used for the production bridge decks, except that the Contractor may elect to finish the test slab with a hand-operated strike-board.

After the construction of the test slab and the demonstration of bridge deck construction operations is complete, the Contractor shall remove and dispose of the test slab in accordance with Sections 3-02.3 and 3-03.3(7)C.

6-02.3(10)D2 Preparation for Concrete Placement

Before placing bridge approach slab concrete, the subgrade shall be constructed in accordance with Sections 3-05 and 5-05.3(6).

Before concrete is placed, the finishing machine shall be operated over the entire length of the deck/slab to check screed deflection. Concrete placement may begin only if the Engineer accepts after this test.

Immediately before placing concrete, the Contractor shall check (and adjust if necessary) all falsework and wedges to minimize settlement and deflection from the added mass of the concrete deck/slab. The Contractor shall also install devices, such as telltales, by which the Engineer can readily measure settlement and deflection.

6-02.3(10)D3 Concrete Placement

The placement operation shall cover the full width of the bridge deck or the full width between construction joints. The Contractor shall locate all construction joints over a beam or web that can support the deck/slab on each side of the joint. The joint shall not occur over a pier unless the Plans permit. Each joint shall be formed vertically and in true alignment. The Contractor shall not release falsework or wedges supporting bridge deck placement sections on either side of a joint until each side has aged as these Specifications require.

Placement of concrete for bridge decks and bridge approach slabs shall comply with Section 6-02.3(6). In placing the concrete, the Contractor shall:

1. Place it (without segregation) against concrete placed earlier, as near as possible to its final position, approximately to grade, and in shallow, closely spaced piles;
2. Consolidate it around reinforcing steel by using vibrators before strike-off by the finishing machine;
3. Not use vibrators to move concrete;
4. Not revibrate the concrete surface areas where workers have stopped prior to screeding;
5. Remove concrete, if any, splashed onto reinforcing steel in adjacent segments before concreting them;
6. Maintain a slight excess of concrete in front of the screed across the entire width of the placement operation. The Contractor shall coordinate the rate of placement such that the concrete is placed, consolidated, and struck off within 30 minutes from the time of placement, unless otherwise accepted by the Engineer at the pre-deck pour meeting;
7. Operate the finishing machine to create a surface that is true and ready for final finish without overfinishing or bringing excessive amounts of mortar to the surface; and
8. Leave a thin, even film of mortar on the concrete surface after the last pass of the finishing machine pan.

Workers shall complete all post screeding operations without walking on the concrete. This may require work bridges spanning the full width of the deck/slab.

After removing the screed supports, the Contractor shall fill the voids with concrete (not mortar).

If the surface left by the finishing machine is porous, rough, or has minor irregularities, the Contractor shall float the surface of the concrete. Floating shall leave a smooth and even surface. Float finishing shall be kept to the minimum number of passes necessary to seal the surface. The floats shall be at least 4-feet long. Each transverse pass of the float shall overlap the previous pass by at least half the length of the float. The first floating shall be at right angles to the strike-off. The second floating shall be at right angles to the centerline of the span. A smooth riding surface shall be maintained across construction joints.

The edge of completed roadway slabs at expansion joints and compression seals shall have a ⅜-inch radius.

After floating, but while the concrete remains plastic, the Contractor shall test the entire deck/slab for flatness (allowing for crown, camber, and vertical curvature). The testing shall be done with a 10-foot straightedge held on the surface. The straightedge shall be advanced in successive positions parallel to the centerline, moving not more than one half the length of the straightedge each time it advances. This procedure shall be repeated with the straightedge held perpendicular to the centerline. An acceptable surface shall be one free from deviations of more than ⅛-inch under the 10-foot straightedge.

If the test reveals depressions, the Contractor shall fill them with freshly mixed concrete, strike off, consolidate, and refinish them. High areas shall be cut down and refinished. Retesting and refinishing shall continue until a surface conforming to the requirements specified above is produced.

6-02.3(10)D4 Vacant

6-02.3(10)D5 Bridge Deck Concrete Finishing and Texturing

Except as otherwise specified for portions of bridge decks receiving an overlay or sidewalk under the same Contract, the Contractor shall texture the surface of the bridge deck as follows:

> The Contractor shall texture the bridge deck using diamond tipped saw blades mounted on a power driven, self-propelled machine that is designed to texture concrete surfaces. The grooving equipment shall provide grooves that are ⅛" ± 1⁄64" wide, 3⁄16" ± 1⁄16" deep, and spaced at ¾" ± ⅛". The bridge deck shall not be textured with a metal tined comb.
>
> The Contractor shall submit a Type 2 Working Drawing consisting of the type of grooving equipment to be used. The Contractor shall demonstrate that the method and equipment for texturing the bridge deck will not chip, spall or otherwise damage the deck.
>
> Unless otherwise allowed by the Engineer, the Contractor shall texture the concrete bridge deck surface either in a longitudinal direction, parallel with centerline or in a transverse direction, perpendicular with centerline. The Contractor shall texture the bridge deck surface to within 3-inches minimum and 24-inches maximum of the edge of concrete at expansion joints, within 1-foot minimum and 2-feet maximum of the curb line, and within 3-inches minimum and 9-inches maximum of the perimeter of bridge drain assemblies.
>
> The Contractor shall contain and collect all concrete dust and debris generated by the bridge deck texturing process, and shall dispose of the collected concrete dust and debris in accordance with Section 3-03.3(7)C.

If the Plans call for placement of a sidewalk or an HMA or concrete overlay on the bridge deck, the Contractor shall produce the final finish of these areas by dragging a strip of damp, seamless burlap lengthwise over the bridge deck or by brooming it lightly. Approximately 3-feet of the drag shall contact the surface, with the least possible bow in its leading edge. It shall be kept wet and free of hardened lumps of concrete. When the burlap drag fails to produce the required finish, the Contractor shall replace it. When not in use, it shall be lifted clear of the bridge deck.

After the bridge deck has cured, the surface shall conform to the surface smoothness requirements specified in Section 6-02.3(10)D3.

Areas identified as not meeting the requirements of Section 6-02.3(10)D3 shall be repaired with a procedure acceptable to the Engineer. All repairs shall be completed prior to surface texturing.

Methods used to remove high spots shall cut through the mortar and aggregate without breaking or dislodging the aggregate or causing spalls.

6-02.3(10)D6 Bridge Approach Slab Finishing and Texturing

Bridge approach slabs that are being built as part of a bridge construction project shall be textured in accordance with Section 6-02.3(10)D5. All other bridge approach slabs shall be textured using metal tined combs in the transverse direction, except bridge approach slabs receiving an overlay in the same Contract shall be finished as specified in Section 6-02.3(10)D5 only.

The comb shall be made of a single row of metal tines. It shall leave striations in the fresh concrete approximately 3⁄16-inch deep by ⅛-inch wide and spaced approximately ½-inch apart. The Engineer will decide actual depths at the site. If the comb has not been accepted, the Contractor shall obtain the Engineer's acceptance by demonstrating it on a test section. The Contractor may operate the combs manually or mechanically, either singly or with several placed end to end. The timing and method used shall produce the required texture without displacing larger particles of aggregate.

Texturing shall end 2-feet from curb lines. This 2-foot untextured strip shall be hand finished with a steel trowel.

Surface smoothness, high spots, and low spots shall be addressed as specified in Section 6-02.3(10)D5. The surface texture on an area cut down or built up shall match closely that of the surrounding bridge approach slab area. The entire bridge approach slab shall provide a smooth riding surface.

6-02.3(10)E Sidewalk

Concrete for sidewalk shall be well compacted, struck off with a strike-board, and floated with a wooden float to achieve a surface that does not vary more than ⅛ inch under a 10-foot straightedge. An edging tool shall be used to finish all sidewalk edges and expansion joints. The final surface shall have a granular texture that will not turn slick when wet.

6-02.3(10)F Bridge Approach Slab Orientation and Anchors

Bridge approach slabs shall be constructed full bridge deck width from outside usable Shoulder to outside usable Shoulder at an elevation to match the Structure. Unless otherwise shown in the Plans, the pavement end of the bridge approach slab shall be constructed normal to the Roadway centerline. The bridge approach slabs shall be modified as shown in the Plans to accommodate the grate inlets at the bridge ends if the grate inlets are required.

Bridge approach slab anchors shall be installed as detailed in the Plans, and the anchor rods, couplers, and nuts shall conform to Section 9-06.5(1). The steel plates shall conform to ASTM A36. All metal parts of the approach expansion anchor shall receive one coat of paint conforming to Section 9-08.1(2)F or be galvanized in accordance with AASHTO M232. The pipe shall be non-perforated PE or PVC pipe of the diameter specified in the Plans. Polystyrene shall conform to Section 9-04.6. The anchors shall be installed parallel both to profile grade and centerline of Roadway. The Contractor shall secure the anchors to ensure that they will not be misaligned during concrete placement. For Method B anchor installations, the epoxy bonding agent used to install the anchors shall be Type IV conforming to Section 9-26.1. The compression seal shall be as noted in the Contract documents. Dowel bars shall be installed in the bridge approach slabs in accordance with the requirements of the *Standard Plans* and Section 5-05.3(10).

The compression seal shall be a 2½ inch wide gland and shall conform to Section 9-04.1(4).

6-02.3(11) Curing Concrete

After placement, concrete surfaces shall be cured as follows:

1. Bridge sidewalks, roofs of cut and cover tunnels – Two coats of curing compound covered by white, reflective type sheeting or continuous wet curing. Curing by either method shall be for at least 10 days.
2. Bridge decks — See Section 6-02.3(11)B.
3. Bridge approach slabs – Two coats of curing compound and continuous wet cure for at least 10 days.
4. Concrete barriers and rail bases – See Section 6-02.3(11)A.
5. All other concrete surfaces – Continuous wet curing for at least 3 days.

When continuous wet curing is specified, the Contractor shall keep all exposed concrete surfaces saturated with water. Formed concrete surfaces shall be kept in a continuous wet cure by leaving the forms in place. If forms are removed during the continuous wet curing period, the Contractor shall treat the concrete as an exposed concrete surface. Runoff water shall be collected and disposed of in accordance with all applicable regulations. In no case shall runoff water be allowed to enter lakes, streams, or other surface waters.

When curing compound is specified, it shall conform to Section 9-23.2. The Contractor shall use white pigmented curing compound (Type 2), unless stated otherwise. If the surface will be covered with HMA, the curing compound shall be white (Type 2, Class B). For bridge sidewalks, the curing compound shall be clear (Type 1, Class B). The compound shall be applied immediately after finishing and shall be agitated thoroughly just before and during application. Application of the second coat shall run at right angles to that of the first, and the coverage shall total at least 1 gallon per 150 square feet. If any curing compound spills on construction joints or reinforcing steel, the Contractor shall remove it before the next concrete placement. If other materials are to be bonded to the surface (e.g., HMA, pigmented sealer), the Contractor shall remove the curing compound by sandblasting or acceptable high-pressure water washing after the curing is completed.

The Contractor shall have on the site, back-up spray equipment, enough workers, and (if needed) a work bridge from which they will apply the curing compound. The Engineer may require the Contractor to demonstrate (at least 1 day before the scheduled concrete placement) that the crew and equipment can apply the compound acceptably.

When white, reflective type sheeting is specified, it shall conform to Section 9-23.1. The sheeting shall be kept in place by taping or weighting the edges where they overlap.

When curing bridge approach slabs, two coats of curing compound shall be applied within 15 minutes after floating and tining any portion of the bridge approach slabs. The continuous wet curing shall be established as soon as the concrete has set enough to allow covering without damaging the finish.

When accelerating admixtures are used, the concrete shall be cured in accordance with these Specifications or until the concrete has reached 70 percent of the mix design 28-day strength, but not less than 3 days.

6-02.3(11)A Curing and Finishing Concrete Barriers and Rail Bases

6-02.3(11)A1 Fixed-Form Barrier

The fixed-form wet curing period shall be at least 10 days.

The edge chamfers shall be formed by attaching chamfer strips to the barrier forms.

After troweling and edging a barrier (while the forms remain in place), the Contractor shall:

1. Brush the top surface with a fine bristle brush;
2. Cover the top surface with heavy, quilted blankets (not burlap); and
3. Spray water on the blankets and forms at intervals sufficient to keep them continuously wet for 3 days minimum. Alternatively, if forms are removed in accordance with Section 6-02.3(17)N, heavy, quilted blankets (not burlap) shall be placed to completely cover the barrier immediately after form removal and kept continuously wet.

After performing the above steps, the Contractor shall:

4. Remove all lips and edgings with sharp tools or chisels;
5. Fill all holes with mortar conforming to Section 9-20.4(2);
6. True up corners of openings;
7. Remove concrete projecting beyond the true surface by stoning or grinding;
8. Completely cover the barrier with heavy, quilted blankets (not burlap);
9. Keep the blankets continuously wet for the remainder of the curing period.

The Contractor may start the finishing Work described in steps 4 through 7 above after the third day of curing if the entire barrier is kept covered except the immediate Work area. Otherwise, no finishing Work may be done until the wet curing period has been completed.

After a minimum of 7 days of the wet curing period, the Contractor shall clean the barrier by removing all form-release agent, mud, dust, other foreign substances, blisters, and air voids just below the surface, to the satisfaction of the Engineer. This shall be accomplished in either of two ways: (1) by light sandblasting and washing with water, or (2) by spraying with a high-pressure water jet. The water jet equipment shall use clean fresh water and shall produce (at the nozzle) at least 1,500 psi with a discharge of at least 3 gpm. The water jet nozzle shall have a 25-degree tip and shall be held no more than 9 inches from the surface being washed.

After cleaning, the Contractor shall use brushes to rub mortar conforming to Section 9-20.4(2) at a ratio of 1:1 cement/aggregate ratio into air holes and small crevices on all surfaces except the brushed top. As soon as the mortar takes its initial set, the Contractor shall rub it off with a piece of sacking or carpet. The barrier shall then be completely covered with wet blankets for the greater of 48 hours or the remainder of the wet curing period.

If the above steps of cleaning and filling with mortar are performed during the 10-day wet curing period, the wet curing blankets shall only be removed in the immediate Work area while the Work is being performed and for no longer than 8 hours.

No curing compound shall be used on fixed-form concrete barrier. The completed surface of the concrete shall be even in color and texture.

6-02.3(11)A2 Slip-Form Barrier

The edge radius shall be formed by attaching radius strips to the barrier slip form.

The Contractor shall finish slip-form barrier by: (1) steel troweling to close all surface pockmarks and holes; and (2) for plain surface barrier, lightly brushing the front and back face with vertical strokes and the top surface with transverse strokes.

After finishing, the Contractor shall cure the slip-form barrier by using either Method A (curing compound) or B (wet blankets) described below.

Method A – Under the curing compound method, the Contractor shall:

1. Spray two coats of clear curing compound (Type 1) on the concrete surface after the free water has disappeared.
2. No later than the morning after applying the curing compound, cover the barrier with white, reflective sheeting for at least 10 days.
3. After the 10-day curing period, remove the curing compound as necessary by light sandblasting or by spraying with a high-pressure water jet to produce an even surface appearance. The water jet equipment shall use clean fresh water and shall produce (at the nozzle) at least 2,500 psi with a discharge of at least 4 gpm. The water jet nozzle shall have a 25-degree tip and shall be held no more than 9 inches from the surface being cleaned. The Contractor may propose to use a curing compound/concrete sealer. The Engineer will evaluate the proposal and if found acceptable, will accept the proposal in writing. As a minimum, the Contractor’s proposal shall include:
    - Product identity
    - Manufacturer’s recommended application rate
    - Method of application and necessary equipment
    - Safety Data Sheet (SDS)
    - Sample of the material for testing

    Allow 14 working days for evaluating the proposal and testing the material.

Method B – Under the wet curing method, the Contractor shall:

1. Provide an initial curing period by continuous fogging or mist spraying for at least the first 24 hours.
2. After the initial curing period, cover the barrier with a heavy, quilted blanket.
3. Keep the blankets continuously wet for at least 10 days. No additional finishing is required at the end of the curing period.

6-02.3(11)B Curing Bridge Deck

6-02.3(11)B1 Equipment & Submittals

The fogging apparatus shall consist of pressure washers with a minimum nozzle output of 1,500 psi, or other means accepted by the Engineer.

The Contractor shall submit a Type 2 Working Drawing consisting of the bridge deck curing plan a minimum of 14 calendar days prior to the pre-deck pour meeting. The Contractor's plan shall describe the sequence and timing that will be used to fog the bridge deck, apply pre-soaked burlap, install soaker hoses and cover the deck with white, reflective sheeting.

6-02.3(11)B2 Curing

The fogging apparatus shall be in place and charged for fogging prior to beginning concrete placement for the bridge deck.

The Contractor shall presoak all burlap to be used to cover the deck during curing.

Immediately after the finishing machine passes over finished concrete, the Contractor shall implement the following tasks:

1. The Contractor shall fog the bridge deck as necessary to suppress evaporation and maintain a wet sheen without developing pooling or sheeting water.
2. The Contractor shall apply the presoaked burlap to the top surface to fully cover the deck within 1 hour after the finishing machine has passed, unless otherwise accepted in the cold weather protection Working Drawing or as accepted by the Engineer during deck casting. The burlap shall be placed flat from work bridges to minimize unnecessary damage to the concrete finish. The Contractor shall not apply curing compound.
3. The Contractor shall continue to keep the burlap wet by fogging as needed until the burlap is covered by soaker hoses and white, reflective sheeting. The Contractor shall place the soaker hoses and white, reflective sheeting after the concrete has achieved initial set. The Contractor shall charge the soaker hoses frequently so as to keep the burlap covering the entire deck wet during the course of curing.

As an alternative to tasks 2 and 3 above, the Contractor may propose a curing system using proprietary curing blankets specifically manufactured for bridge deck curing. The Contractor shall submit a Type 2 Working Drawing consisting of details of the proprietary curing blanket system, including product literature and details of how the system is to be installed and maintained.

The wet curing regime as described shall remain in place for at least 14 consecutive calendar days.

6-02.3(12) Construction Joints

6-02.3(12)A Construction Joints in New Construction

If the Engineer allows, the Contractor may add, delete, or relocate construction joints shown in the Plans. All requests for such changes shall be in writing, accompanied by a drawing that depicts them and shall be submitted as an RFI in accordance with Section 1-05.1(2). The Contractor will bear all added costs that result from such changes.

All construction joints shall be formed neatly with grade strips or other accepted methods. The Contracting Agency will not accept irregular or wavy pour lines. All joints shall be horizontal, vertical, or perpendicular to the main reinforcement. The Contractor shall not use an edger on construction joints, and shall remove all lips or edging before making the adjacent pour.

If the Plans require a roughened surface on the joint, the Contractor shall strike it off to leave grooves at right angles to the length of the member. Grooves shall be installed using one of the following options:

1. Grooves shall be ½ to 1 inch wide, ¼ to ½ inch deep, and spaced equally at twice the width of the groove. Grooves shall terminate approximately 1½-inches from the face of concrete.
2. Grooves shall be 1 to 2 inches wide, a minimum of ½-inch deep, and spaced a maximum of three times the width of the groove. Grooves shall terminate approximately 1½-inches from the face of concrete.

If the Engineer allows, the Contractor may use an alternate method to produce a roughened surface on the joint, provided that such an alternate method leaves a roughened surface of at least a ¼-inch amplitude.

If the first strike-off does not produce the required roughness, the Contractor shall repeat the process before the concrete reaches initial set. The final surface shall be clean and without laitance or loose material.

If the Plans do not require a roughened surface, the Contractor shall include shear keys at all construction joints. These keys shall provide a positive, mechanical bond. Shear keys shall be formed depressions and the forms shall not be removed until the concrete has been in place at least 12 hours. Forms shall be slightly beveled to ensure ready removal. Raised shear keys are not allowed.

Shear keys for the tops of beams, at tops and bottoms of boxed girder webs, in diaphragms, and in crossbeams shall:

1. Be formed with 2 by 8-inch wood blocks;
2. Measure 8 inches lengthwise along the beam or girder stem;
3. Measure 4 inches less than the width of the stem, beam, crossbeam, etc. (measured transverse of the stem); and
4. Be spaced at 16 inches center to center.

Unless the Plans show otherwise, in other locations (not named above), shear keys shall equal approximately ⅓ of the joint area and shall be approximately 1½ inches deep.

Before placing fresh concrete against cured concrete, the Contractor shall thoroughly clean and saturate the cured surface. All loose particles, dust, dirt, laitance, oil, or film of any sort shall be removed by method(s) as accepted by the Engineer. The cleaned surface shall be saturated with water for a minimum of four hours before the fresh concrete is placed.

Before placing the reinforcing mat for footings on seals, the Contractor shall: (1) remove all scum, laitance, and loose gravel and sediment; (2) clean the construction joint at the top of the seals; and (3) chip off all high spots on the seals that would prevent the footing steel from being placed in the position required by the Plans.

6-02.3(12)B Construction Joints Between Existing and New Construction

If the Plans or Special Provisions require a roughened surface on the joint, the Contractor shall thoroughly roughen the existing surface to a uniformly distributed ¼-inch minimum amplitude surface profile, with peaks spaced at a maximum of 1 inch. These requirements do not apply to construction joints below the top of drilled shafts.

If the Plans or Special Provisions do not require a roughened surface on the joint, the Contractor shall remove all loose particles, dust, dirt, laitance, oil, or film of any sort.

Before placing fresh concrete against existing concrete, the Contractor shall thoroughly clean and saturate the existing surface. All loose particles, dust, dirt, laitance, oil, or film of any sort shall be removed. The cleaned surface shall be saturated with water for a minimum of 4 hours before the fresh concrete is placed.

6-02.3(13) Expansion Joints

This section outlines the requirements of specific expansion joints shown in the Plans. The Plans may require other types of joints, seals, or materials than those described here.

Joints made of a vulcanized, elastomeric compound (with neoprene as the only polymer) shall be installed with a lubricant adhesive as recommended by the manufacturer. The length of a seal shall match that required in the Plans without splicing or stretching.

Open joints shall be formed with a template made of wood, metal, or other suitable material. Insertion and removal of the template shall be done without chipping or breaking the edges or otherwise damaging the concrete.

Any part of an expansion joint running parallel to the direction of expansion shall provide a clearance of at least ½ inch (produced by inserting and removing a spacer strip) between the two surfaces. The Contractor shall ensure that the surfaces are precisely parallel to prevent wedging from expansion and contraction.

All poured rubber joint sealer (and any required primer) shall conform with Section 9-04.2(2).

6-02.3(13)A Strip Seal Expansion Joint System

The Contractor shall submit Type 2 Working Drawings consisting of the strip seal expansion joint shop drawings. These plans shall include, at a minimum, the following:

1. Plan, elevation, and sections of the joint system and all components, with dimensions and tolerances.
2. All material designations.
3. Manufacturer's written installation procedure. The installation procedure shall indicate how the extrusions set into the two sides of the joint will be allowed to move independently of one another.
4. Corrosion protection system used on the metal components.
5. Locations of welded shear studs, lifting mechanisms, temperature setting devices, and construction adjustment devices.
6. Method of sealing the system to prevent leakage of water through the joint.
7. Details of the temporary supports for the steel extrusions while the encapsulating concrete of the headers is placed and cured.
8. The gland installation procedure, including the means and methods used to install the gland and assure correct seating of the gland within the steel extrusions.

The strip seal shall be removable and replaceable.

The metal components shall conform to ASTM A36, ASTM A992, or ASTM A572, and shall be protected against corrosion by one of the following methods:

1. Zinc metallized in accordance with Section 6-07.3(14).
2. Hot-dip galvanized in accordance with AASHTO M111.
3. Paint in accordance with Section 6-07.3(9). The color of the top coat shall be SAE AMS Standard 595 Color No. 26357. The surfaces embedded in concrete shall be painted only with a shop primer coat of paint conforming to Section 9-08.1(2)C.

If the gland is installed in the field, the Contractor shall have the services of a strip seal expansion joint system manufacturer's technical representative physically present at the job site. The manufacturer's technical representative shall train the Contractor's personnel performing the field installation of the gland, provide technical assistance for installing the gland, and observe and inspect the installation of at least the first complete joint.

The strip seal gland shall be continuous for the full length of the joint with no splices permitted, unless otherwise shown in the Plans.

Other than items shown in the Plans, threaded studs used for construction adjustments are the only items that may be welded to the steel shapes provided they are removed by grinding after use, and the area repaired by application of an accepted corrosion protection system.

After the expansion joint system is installed, a watertightness test shall be performed as follows. The Contractor shall flood each completely installed expansion joint system with water to a minimum depth of three inches for a duration of at least one hour. If leakage is observed, the expansion joint system shall be repaired at no additional expense to the Contracting Agency, as recommended by the manufacturer. After repairs are completed, the expansion joint shall be retested for leakage.

6-02.3(13)B Compression Seal Expansion Joint System

Compression seal glands shall conform to Section 9-04.1(4) and be sized as shown in the Plans.

The compression seal expansion joint system shall be installed in accordance with the manufacturer's written recommendations. The Contractor shall submit a Type 1 Working Drawing consisting of the manufacturer's written installation procedure and repair procedures if leakage testing fails.

After the expansion joint system is installed, a watertightness test shall be performed as follows. The Contractor shall flood each completely installed expansion joint system with water to a minimum depth of three inches for a duration of at least one hour. If leakage is observed, the expansion joint system shall be repaired at no additional expense to the Contracting Agency, as recommended by the manufacturer. After repairs are completed, the expansion joint shall be retested for leakage.

6-02.3(13)C Modular Expansion Joint System

The Contractor shall design, fabricate, inspect, test, and install a modular, multiple seal expansion joint system in accordance with the geometry and movements shown and specified in the Plans. The modular expansion joint system shall extend continuously across the full width of the bridge deck and up into the traffic barriers as shown in the Plans.

6-02.3(13)C1 Acceptable Manufacturers

Only manufacturers whose modular expansion joint systems have met the requirements specified in Section 6-02.3(13)C9 will be permitted to supply modular expansion joint systems. Testing required to establish the fatigue resistance of all details of a specific proprietary system shall be completed prior to the Contract Award date. All fatigue testing shall be conducted in accordance with Sections 6-02.3(13)C11, 6-02.3(13)C23, and 6-02.3(13)C26. Testing shall be completed on all revised details or material substitutions of a previously prequalified system prior to the Contract award date.

Manufacturers known to have met the requirements of Section 6-02.3(13)C9 are specified in Section 6-02.3(13)C as supplemented in the Special Provisions.

6-02.3(13)C2 Submittals

The expansion joint manufacturer shall have at least three years of experience in designing and manufacturing modular expansion joint systems. The Contractor shall submit a Type 1 Working Drawing consisting of written certification of the manufacturer's experience, including the location of each bridge, installation date, governmental agency/owner, and the name, address, and telephone number of each owner's/agency's representative.

The Contractor shall submit the name of the selected expansion joint system manufacturer to the Engineer within 10 days of Contract award. Once the name of the manufacturer has been submitted to the Engineer, the Contractor shall not select an alternative expansion joint system manufacturer unless the manufacturer demonstrates an inability to meet the requirements of Section 6-02.3(13)C.

The Contractor shall submit Type 3E Working Drawings consisting of shop drawings and design calculations delineating the expansion joint system in accordance with Sections 1-05.3 and 6-03.3(7) and as noted herein. The Professional Engineer responsible for preparing and stamping the submittal shall be an employee of the expansion joint system manufacturer, and shall hold a valid license in the branch of Civil or Structural Engineering, either in the State of Washington or another state. These submittals shall include, at a minimum, the following:

1. Plan, elevation, and section of the joint system for each movement rating and bridge deck width. All dimensions and tolerances shall be specified.
2. Sections showing all materials composing the expansion joint system with complete details of all individual components including all bolted and welded splices and connections.
3. All ASTM, AASHTO, or other material designations.
4. Installation plan including sequence, lifting mechanisms and locations, details of temporary anchorage during setting, temperature adjustment devices, opening dimensions relative to temperature, installation details at curbs, and seal installation details.
5. Plan for achieving watertightness including details related to performing the watertightness test required in Section 6-02.3(13)C32.
6. Details and material designations pertinent to the corrosion protection system.
7. Requirements and details related to the temporary support of the joint system for shipping, handling, and job site storage.
8. Design calculations for all structural elements including all springs and bearings. The design calculations shall include fatigue design for all structural elements, connections, and splices.
9. Welding procedures in compliance with the current AASHTO/AWS D1.5 Bridge Welding Code.
10. A written maintenance and part replacement plan to facilitate replacement of parts subject to wear. This plan shall include a list of parts, instructions for maintenance inspection, acceptable wear tolerances, methods for determining wear, procedures for replacing worn parts, and procedures for replacing seals.
11. Comprehensive integrated details of the expansion joint system, its support boxes, assembly supports, erection aids, and the bridge deck and expansion joint header steel reinforcing bars. The Contractor shall identify in the integrated details all modifications to the bridge deck steel reinforcing bars necessary to accommodate the expansion joint system. The Contractor shall show, in the integrated details, the specific means (moving, bending, cutting, bundling, supplementing or coupling steel

reinforcing bars, or incorporating hooks or headed steel reinforcing bars) to address congestion and conflicts.

12. Means, methods, and concrete placement sequence for placing concrete and attaining full consolidation of concrete beneath and adjacent to the support boxes of the modular expansion joint assembly. The methods and sequence shall account for congestion surrounding the box sections due to bridge deck steel reinforcing bars, and expansion joint assembly supports and erection aids.

At the time of shop plan submittal as outlined above, the Contractor shall submit Type 1 Working Drawings consisting of the following documentation:

1. Documentation that the manufacturer is certified through the AISC Quality Certification Program under the category Bridge and Highway Metal Components.
2. Documentation that welding inspection personnel are qualified and certified as welding inspectors under AWS QC1, Standard for Qualification and Certification of Welding Inspectors.
3. Documentation that personnel performing nondestructive testing (NDT) are qualified and certified as NDT Level II under the American Society for Nondestructive Testing (ASNT) Recommended Practice SNT-TC-1a.

The Contractor shall submit Type 1 Working Drawings consisting of the following test reports and certificates of compliance:

1. Manufacturer's certificate of compliance for all polytetrafluorethylene (PTFE) sheeting, PTFE fabric, and elastomer.
2. Certified mill test reports for all steel and stainless steel in the expansion joint system assemblies.
3. Certified test reports confirming that the springs and bearings meet the design load requirements.

Upon completion of installation, the Contractor shall submit a Type 1 Working Drawing consisting of certification stating that each expansion joint system was installed in accordance with the shop plan installation procedure. This certification shall conform to the requirements specified in Section 6-02.3(13)C32.

The Contractor shall submit Type 2E Working Drawings consisting of a temporary bridging method for each expansion joint system over which construction traffic is anticipated to cross following its installation. This submittal shall conform to the requirements specified in Section 6-02.3(13)C32.

The Contractor shall submit Type 1 Working Drawings consisting of a Quality Assurance Inspection program performed by an independent inspection agency provided by the manufacturer. The name of the independent inspection agency, details of the proposed quality assurance inspection program including inspection frequency, and all applicable reporting forms shall be included in the Type 1 Working Drawing submittal.

Modular expansion joint assembly warranties and guarantees provided by the manufacturer in accordance with Section 1-05.10 shall be submitted as Type 1 Working Drawings.

6-02.3(13)C3 General Design Requirements

The expansion joint system shall be designed and detailed with adequate access to all internal components in order to assure the feasibility of inspection and maintenance activities.

The expansion joint system shall be designed and detailed to minimize concrete cracking above the support boxes. Measures taken shall include, but not be limited to, assuring adequate support box top plate thickness, specifying any additional bridge deck steel reinforcement required, and providing adequate concrete cover.

The expansion joint system and bridge deck steel reinforcement shall be detailed to assure that adequate concrete consolidation can be achieved underneath all support boxes.

The expansion joint seals shall not protrude above the top of the expansion joint system under any service condition. Split extrusions may be used at curb upturns.

The elastomeric or urethane springs and bearings shall be designed to be removable and replaceable. The removal and reinstallation of each strip seal shall be easily accomplished from above the joint with a 1-¼ inch minimum gap width. These operations shall be viable with a one lane partial closure of the bridge deck.

The expansion joint system shall be designed and detailed to be watertight.

The expansion joint system shall be designed and detailed to accommodate all movements specified in the Plans.

The expansion joint shall be designed and detailed to mitigate the potential for fatigue damage wherever centerbeam field splices are required. Consideration shall be given to reducing support box spacing and optimizing splice location between adjacent support boxes in order to minimize fatigue stress range at field splices.

6-02.3(13)C4 Design Axle Loads and Impact Factors

The centerbeams, support bars, bearings, connections, and other structural components shall be designed for the simultaneous application of vertical and horizontal loads from a tandem axle. The tandem axle shall consist of a pair of axles spaced four feet apart with vertical and horizontal loads as specified in Section 6-02.3(13)C as supplemented in the Special Provisions. The transverse spacing of the wheels shall be six feet. The distribution of the wheel load among centerbeams shall be as specified in Section 6-02.3(13)C5.

6-02.3(13)C5 Distribution of Wheel Loads

The following table specifies the centerbeam distribution factor as a function of centerbeam top flange width. This factor is the percentage of the design vertical axle load and the design horizontal axle load that shall be applied to an individual centerbeam for the design of that centerbeam and its associated support bars. Distribution factors shall be interpolated for centerbeam top flange widths between those explicitly denoted in the table. In no case shall the distribution factor be taken as less than 50%. The remainder of the load shall be divided equally and applied to the two adjacent centerbeams or edge beams.

Width of Centerbeam Top Flange	Distribution Factor
2.5 inches	50%
3.0 inches	60%
4.0 inches	70%
4.75 inches	80%

6-02.3(13)C6 Fatigue Limit State Design Requirements

Modular expansion joint system structural members, bolted and welded splices and connections, and attachments shall be designed to resist the Fatigue Limit State load combination specified in Table 3.4.1-1 of the AASHTO LRFD Bridge Design Specifications. The vertical and horizontal load ranges specified in Section 6-02.3(13)C4 shall be applied simultaneously. These loads shall be distributed as specified in Section 6-02.3(13)C5.

The nominal stress ranges, Δf, at all fatigue critical details shall be obtained from a structural analysis of the expansion joint system applying the design vertical and horizontal load ranges specified in Section 6-02.3(13)C4 and distributed as specified in Section 6-02.3(13)C5. The expansion joint system shall be analyzed with a minimum gap opening corresponding to the midrange configuration (at least half of the maximum gap opening). The design axle load shall be applied as two wheel loads, each having a transverse width of 20 inches.

For each detail under consideration, the wheel loads shall be positioned transversely on a centerbeam to achieve the maximum nominal stress range at that detail. The vertical and horizontal wheel loads shall be applied as line loads to the top of the centerbeams at their centerlines. The design stress range in the centerbeam-to-support bar connection shall be calculated as specified below. The design nominal stress ranges, Δf, multiplied by the appropriate load factors in Table 3.4.1-1 of the AASHTO LRFD Bridge Design Specifications, shall be used for fatigue design as specified at the end of this subsection.

6-02.3(13)C7 Welded or Bolted Single-Support-Bar Systems

The nominal stress range, Δf, in the centerbeam at a welded or bolted stirrup shall be the sum of the longitudinal bending stress ranges at the critical section resulting from vertical and horizontal loading. The effects of stresses in any load-bearing attachments such as the stirrup or yoke shall not be considered when calculating the longitudinal stress range in the centerbeam. For bolted single-support-bar systems, stress ranges shall be calculated using the net section.

The nominal stress range, Δf, in the stirrup or yoke shall be calculated without considering the effects of stresses in the centerbeam. The stress range shall be calculated by assuming a load range in the stirrup equal to 30% of the total vertical reaction force between the centerbeam and the support bar. The effects of horizontal loads may be neglected in the design of the stirrup.

6-02.3(13)C8 Welded Multiple-Support-Bar Systems

Three locations have been identified as initiation sites for fatigue cracking at a centerbeam-to-support bar welded connection. The types of cracking associated with these three locations are described below. The corresponding equations may be used to calculate the nominal stress range, Δf. For the support bar, either the reduced moment at the critical cross section or the moment at the centerline of the connection may be used in these equations.

Centerbeam weld toe cracking is driven by a combination of longitudinal bending stress range, S_{RB}, in the centerbeam, and vertical stress range, S_{RZ}, at the top of the connection weld.

The longitudinal bending stress range, S_{RB}, at the bottom of the centerbeam shall be calculated as:

$$S_{RB} = M_{Vcb} / S_{Xcb} + M_{Hcb} / S_{Ycb}$$

The vertical stress range, S_{RZ}, at the top of the connection weld shall be calculated as:

$$S_{RZ} = R_H \cdot d_{cb} / S_{Wtop} + R_V / A_{Wtop}$$

Support bar weld toe cracking is driven by a combination of longitudinal bending stress range, S_{RB}, in the support bar and vertical stress range, S_{RZ}, at the bottom of the connection weld.

The longitudinal bending stress range, S_{RB}, at the top of the support bar shall be calculated as:

$$S_{RB} = M_{Vsb} / S_{Xsb} + 0.5 \cdot R_H \cdot (d_{cb} + h_W + 0.5 \cdot d_{sb}) / S_{Xsb}$$

The vertical stress range, S_{RZ}, at the bottom of the connection weld shall be calculated as:

$$S_{RZ} = R_H \cdot (d_{cb} + h_W) / S_{wbot} + R_V / A_{Wbot}$$

Weld throat cracking is driven by a vertical stress range at the weld throat.

The vertical stress range, S_{RZ}, at mid-height of the connection weld shall be calculated as:

$$S_{RZ} = R_V / A_{wmid} + R_H \cdot (d_{cb} + 0.5 \cdot h_W) / S_{Wmid}$$

In the above equations:

R_V	=	vertical reaction at the connection weld
R_H	=	horizontal reaction at the connection weld
M_{Vcb}	=	bending moment in the centerbeam due to applied vertical forces
M_{Hcb}	=	bending moment in the centerbeam due to applied horizontal forces
M_{Vsb}	=	bending moment in the support bar due to applied vertical forces
S_{Xcb}	=	section modulus at bottom of the centerbeam about horizontal axis
S_{Ycb}	=	section modulus of the centerbeam about vertical axis
S_{Xsb}	=	section modulus at top of the support bar about horizontal axis
A_{Wtop}	=	area of the weld at the top of the connection
A_{Wmid}	=	area of the weld at the middle of the connection
A_{Wbot}	=	area of the weld at the bottom of the connection
S_{Wtop}	=	section modulus of the weld at the top of the connection
S_{Wmid}	=	section modulus of the weld at the middle of the connection
S_{Wbot}	=	section modulus of the weld at the bottom of the connection
h_W	=	height of the weld
d_{cb}	=	depth of the centerbeam
d_{sb}	=	depth of the support bar

The nominal stress range, Δf, at welded multiple-support-bar connection details shall be calculated for each case above as follows:

$$\Delta f = (S^2_{RB} + S^2_{RZ})^{1/2}$$

Where:

S_{RB}	=	longitudinal stress range in the centerbeam or support bar, as calculated for each specific case above.
S_{RZ}	=	vertical stress range in the centerbeam-to-support bar connection weld, as calculated for each specific case above.

All modular expansion joint system structural members, connections (bolted and welded), splices, and attachments shall satisfy the following:

$$\gamma \Delta f = (\Delta F)_{TH}$$

Where:

γ	=	the load factor for the Fatigue I Limit State, as stipulated in Table 3.4.1-1 of the AASHTO LRFD Bridge Design Specifications.
Δf	=	the nominal stress range as specified at the beginning of this subsection.
$(\Delta F)_{TH}$	=	constant amplitude fatigue threshold (CAFL) as specified in Section 6-02.3(13)C9.

6-02.3(13)C9 Fatigue Resistance Characterization Requirements

The fatigue resistance of all details shall be characterized in terms of the detail categories specified in Table 6.6.1.2.5-1 of the AASHTO LRFD Bridge Design Specifications.

Many details composing modular expansion joint systems may clearly correspond to specific structural details depicted in Figure 6.6.1.2.3-1 of the AASHTO LRFD Bridge Design Specifications. In these cases, the applicable fatigue categories specified in Table 6.6.1.2.3-1 may be used for design.

In cases where the Engineer establishes that a detail does not clearly correspond to a structural detail depicted in Figure 6.6.1.2.3-1, fatigue testing of specimens exhibiting that detail shall be conducted, in accordance with Sections 6-02.3(13)C11, 6-02.3(13)C23, and 6-02.3(13)C26, to establish the appropriate constant amplitude fatigue limit (CAFL) for that detail.

6-02.3(13)C10 Strength I Limit State Design Requirements

Modular expansion joint system structural steel members, connections (bolted and welded), splices, and attachments shall be designed to resist the Strength I Limit State load combination specified in Table 3.4.1-1 of the AASHTO LRFD Bridge Design Specifications. The vertical and horizontal loads specified in Section 6-02.3(13)C4 shall be applied simultaneously. These loads shall be distributed as specified in Section 6-02.3(13)C5.

6-02.3(13)C11 Fatigue Testing of Metallic Structural Components and Connections

This test procedure is acceptable for, and specifically applicable to, establishing the fatigue resistance of the centerbeam-to-support bar connection in modular expansion joint systems. It is applicable to single-support-bar and multiple-support-bar systems having either welded or bolted centerbeam-to-support bar connections. The same methodology may be applied to establish the fatigue resistance of other modular expansion joint metallic structural component details, including centerbeam splices.

Each fatigue test generates a discrete datum. Each datum comprises an applied constant amplitude nominal stress range, S_r, and the corresponding number of cycles, N, associated with either a predetermined extent of crack propagation, defined as failure, or with termination of the test, defined as runout. Ten data shall be acquired for each connection detail. All data shall be in the very long life range, corresponding as closely to the constant amplitude fatigue limit (CAFL) as practical. Specifically, the number of cycles, N, associated with each datum, shall be no less than one order of magnitude less than Nmin corresponding to the detail category specific CAFL specified in Section 6-02.3(13)C19. For example, to characterize a detail as Detail Category C, the tested number of cycles, N, shall exceed 4.4 x 105 for each datum.

The constant amplitude nominal stress range shall be calculated at the anticipated initiation location of an incipient crack. Nominal stresses shall be calculated using conventional equations for analyzing bending and axial load. These equations are essentially the same as those used in strength design. The stress concentration effects of a weld, bolt hole, or other local features are not explicitly embodied in the conventional nominal stress equations.

The appropriate AASHTO detail category applicable to fatigue design shall be established by comparing acquired test data to fatigue resistance graphs representing the AASHTO detail categories. The constant amplitude fatigue limit (CAFL) applicable to fatigue design corresponds to the AASHTO detail category fatigue resistance graph representing a lower bound of the experimentally acquired data.

When testing is conducted exclusively in the infinite life regime and more stringent test data scatter requirements are satisfied, a unique CAFL (different from those CAFL corresponding to specific detail categories specified by AASHTO) may be established for fatigue design.

Specimens selected for testing shall be full-scale centerbeam and support bar assemblies or subassemblies representative of those installed in field applications. A subassembly is defined as a specimen having the same physical and geometric properties as an assembly but having a reduced number of centerbeams.

Each specimen shall consist of three continuous centerbeam spans over four equally spaced support bars. Centerbeam spans between adjacent support bar centerlines shall be a minimum of 3'-0" and a maximum of 4'-6". Support bar spans shall be a minimum of 3'-0" and a maximum of 3'-8". The centerbeam-to-support bar connection being tested shall be located at the midspan of each support bar.

All welded or bolted attachments used to secure equidistant springs to a support bar, centerbeam, or stirrup shall be fabricated as an integral part of the specimen. A rigid load path to the test fixture shall be provided to resist any horizontal forces or displacements which would normally be resisted through these attachments in a field installation. All miscellaneous welded or bolted attachments, including welded attachments used to secure the expansion joint strip seals to the centerbeams, shall also be fabricated as integral parts of the specimen.

Support bars of subassembly specimens that are components of single-support-bar swivel-joist type modular expansion joint systems shall be oriented perpendicular to the longitudinal axis of the centerbeam.

Prior to testing, each specimen shall be visually inspected for defects, loose fasteners or other aberrations which could plausibly affect the tested fatigue resistance. Defects and flaws shall be defined in accordance with the appropriate governing specification (ASTM A6, AWS D1.5, etc.). Data acquired from specimens containing such anomalies shall not be excluded from consideration except as permitted in Section 6-02.3(13)C20. Observed anomalies shall also be reported with its corresponding data in the tabular format stipulated in Section 6-02.3(13)C22.

Each specimen shall be sufficiently instrumented to measure the static nominal strain range within that specimen for a specific applied load range. Best results can generally be obtained when the applied load range for the static calibration tests does not pass through zero load. Strain measurements shall be made at locations sufficiently distant from local effects, such as weld toes or bolt holes, which could significantly influence acquired test data.

As a minimum, eight strain gages shall be installed on the centerbeam top flange in the vicinity of each centerbeam-to-support bar connection. These gages shall be installed in pairs on each side of the connection at distances of one and two times the depth of the centerbeam from the centerline of the connection. Each pair of strain gages shall be located symmetrically about the centerline of the centerbeam. As a minimum, two strain gages shall also be installed on the support bar bottom flange in the vicinity of each centerbeam-to-support bar connection. One of these strain gages shall be installed on each side of the connection at a distance equal to the depth of the support bar from the centerline of the connection. These strain gages shall be installed along the centerline of the support bar.

6-02.3(13)C12 Fatigue Testing Test Fixtures

Test fixtures shall have the capability to adequately support and secure the specimen throughout the duration of the test. The fixture shall be designed and fabricated to such tolerances as required to assure that additional stresses will not be generated in the specimen as a consequence of fixture misalignment. Mismatches resulting from specimen fabrication errors shall be accommodated by shimming or other such means precluding the application of force to the specimen.

Typical elastomeric bearings and springs used to transfer vertical loads from the support bars to the support boxes may be replaced with steel bearings in the test fixture. This modification will enable fatigue testing at higher load ranges and different frequencies than those encountered during normal service conditions.

Load shall be applied through two 10-inch long patches. Each patch shall typically comprise a steel plate and a hard rubber bearing pad placed in contact with the bottom flange of the centerbeam. Each patch shall be located at midspan of each outer span.

In order to assure adequate seating of the specimen to the test fixture, a minimum of 10 kips shall be applied at each patch location. This requirement is waived for tests of single support bar systems conducted using load reversal. Once this load has been applied, all strain measuring devices shall be rebalanced to zero strain while the preload is maintained. An additional load approximately equivalent to the calculated load range shall be applied. Strain ranges shall be measured for the load range from 10 kips to the peak load. Each static calibration test shall be repeated three times while still maintaining a minimum 10 kips load at each load patch. The measured strain ranges from each repetition should vary by no more than 25% from the mean value. If the stress ranges are not repeatable, appropriate modifications shall be made to the test fixture.

6-02.3(13)C13 Static Calibration Test

Prior to fatigue resistance testing, a static calibration test shall be performed in order to validate the structural analysis model. The static calibration test shall be performed after attainment of stress range repeatability as described in Section 6-02.3(13)C12. The structural analysis model shall be considered validated when calculated strain ranges are within ±25% of the measured strain ranges at every strain gage location.

For the purpose of reporting nominal fatigue resistance stress ranges at specific details, stress ranges determined through structural analysis of the model shall be preferred over stress ranges acquired directly from test measurements.

6-02.3(13)C14 Fatigue Test Procedure

A minimum of ten data points shall be required to establish the fatigue resistance of each detail. The centerbeam-to-support bar connection shall be considered as a single detail.

Several data points may be obtained from a single specimen by repairing the cracked sections of that specimen and resuming testing. Such repairs shall have minimal effect on the stress ranges at unfailed details still being tested. Data points derived from tests in which a repaired detail cracks again shall be discarded.

All data shall be in the very long life range, corresponding as closely to the constant amplitude fatigue limit as practical, but in no case less than 200,000 cycles. Either finite life regime or infinite life regime testing may be conducted. For infinite life regime testing, the number of cycles, N, associated with each of the ten data shall be at least twice the number of cycles, N_{min}, designated in the table in Section 6-02.3(13)C19.

Loads shall be applied using hydraulic actuators or other similar loading devices. The magnitude of the vertical load range, ΔP_v, shall be maintained and continuously monitored throughout the duration of the test. Vertical and horizontal load ranges shall be applied to the specimen simultaneously. The horizontal load range shall always be equal to 20% of the vertical load range, ΔP_v. This horizontal-to-vertical load ratio may be maintained by inclining the specimen 11.3 degrees with respect to the horizontal plane and applying load through vertically oriented actuators.

For multiple support bar systems, the loading mechanism shall be either exclusively tension or exclusively compression and shall be applied at a constant amplitude at any desired frequency. The applied load range shall be in a direction such that the reaction force between the centerbeam and support bar is always tensile. The load range shall not pass through zero load. Minimum preload shall be maintained throughout the duration of the test.

Single support bar systems may be loaded using the same procedures as those for multiple support bar systems. If premature stirrup failure occurs, an applied load range of 70% compression and 30% tension may be used.

The load ranges used in the test shall not be so large as to alter the observed failure mode from that which would be observed under service conditions. Under no circumstance shall imposed stress exceed the yield stress of the material in any portion of the specimen. Each specimen shall be tested using at least two different load (stress) ranges.

If infinite life regime testing is conducted, the first load range should be chosen so that the applied stress range is just above the postulated CAFL. The load range in the subsequent test shall be decreased if failure resulted and increased if the test resulted in a runout. A suggested increment in load is such that the stress range is increased or decreased by 2 ksi. The applicable CAFL shall be selected from those CAFL values corresponding to the AASHTO fatigue categories. The selected CAFL is the one just below the lowest stress range that resulted in cracking.

6-02.3(13)C15 Fatigue Test Failure Criteria

Failure in welded centerbeam-to-support bar connection specimens includes the following:

> Centerbeam weld toe cracking originates at or near the centerbeam weld toe, propagates up into the centerbeam at some angle, and grows back over the connection. These cracks typically grow at an angle of about 45 degrees. A specimen shall be considered as failed due to this type of cracking when the crack has grown on any vertical face a length from the point of origin equal to half of the centerbeam depth.
>
> Support bar weld toe cracking originates at or near the support bar weld toe, propagates down into the support bar, and grows back under the connection at some angle, typically about 45 degrees. A specimen shall be considered as failed due to this type of cracking when the crack has grown on any vertical support bar face a length from the point of origin equal to half of the depth of the support bar.
>
> Weld throat cracking originates in the weld throat and typically grows in a plane parallel to the longitudinal axis of the support bar at about mid-depth of the weld throat. A specimen shall be considered as failed due to this type of cracking when a complete fracture of the weld throat has occurred. These cracks have been observed to turn down into the support bar, but only after significant growth. In such instances, the criteria for support bar weld toe cracking shall be applied.

A welded stirrup connection specimen shall be considered as failed when cracks result in the complete fracture of a stirrup leg or when cracks originating at or near a stirrup weld have grown into any face of the centerbeam a length from the stirrup weld toe equal to half of the centerbeam depth.

A bolted centerbeam-to-support bar connection specimen shall be considered as failed when:

1. Fatigue cracks which have grown out of a bolt hole have resulted in the complete fracture of the tension flange of the centerbeam.
2. Fatigue cracks which have grown out of a bolt hole have extended into any face of the centerbeam web a distance equivalent to half of the centerbeam depth less the centerbeam flange thickness.
3. A portion of a stirrup fractures completely.
4. A single bolt fractures completely.

6-02.3(13)C16 Alternate Criteria for Termination of a Finite Life Regime Fatigue Test

A test may also be terminated when, for a given stress range, the specimen has survived the number of cycles required to plot the data above either a particular fatigue resistance curve or the maximum permitted in Section 6-02.3(13)C20. For example, if the applied stress range is 17 ksi and the desired fatigue resistance curve is Category C, then based upon the equation presented in Section 6-02.3(13)C19, the test may be terminated after application of about 900,000 cycles provided that the specimen has not failed based on the above described criteria.

6-02.3(13)C17 Nominal Stress Range for Welded Centerbeam-to-Support Bar Systems

The nominal stress range for centerbeam weld toe cracking shall be calculated by taking the square root of the sum of the squares of the longitudinal bending stress range in the centerbeam and the vertical stress range at the top of the weld.

The nominal stress range for support bar weld toe cracking shall be calculated by taking the square root of the sum of the squares of the longitudinal bending stress range in the support bar and the vertical stress range at the bottom of the weld.

The nominal stress range for weld throat cracking shall be the calculated vertical stress range in the throat of the weld.

The nominal stress range in the centerbeam at a welded stirrup shall be calculated as the summation of the longitudinal bending stress ranges at the critical section resulting from vertical and horizontal loading. The entire load range shall be used in the calculation, even if the loading is partly in compression. The effects of stresses in the load-bearing attachments such as the stirrup or yoke shall not be considered when calculating the nominal stress range in the centerbeam.

The load range in the stirrup itself shall be taken as 30% of the total vertical load range carried through the connection. The effect of horizontal forces may be neglected.

6-02.3(13)C18 Nominal Stress Range for Bolted Centerbeam-to-Support Bar Systems

The nominal stress range in the centerbeam shall be taken as the summation of the longitudinal bending stress ranges in the centerbeam resulting from vertical and horizontal loading. Nominal stress ranges shall be calculated using the net section. The effects of stresses in the stirrup shall not be considered when calculating the nominal stress range in the centerbeam.

The nominal load range in the bolt group and the stirrup assembly shall be taken as 30% of the total vertical load range carried through the connection. The effect of horizontal forces may be neglected.

6-02.3(13)C19 Interpretation of Fatigue Test Data

The experimentally acquired data and graphs representing the fatigue resistance of the detail categories delineated in Section 6.6 of the AASHTO LRFD Bridge Design Specifications, shall be juxtaposed on a log-log scale. The equation representing the finite life fatigue resistance of these AASHTO detail categories is:

$$N = A / S^3_{r,eff}$$

Where:

N	=	number of cycles to failure.
$S_{r,eff}$	=	nominal effective stress range representing fatigue resistance.
A	=	constant defined in Table 6.6.1.2.5-1 of the AASHTO LRFD Bridge Design Specifications.

The minimum number of cycles associated with infinite fatigue life, N_{min}, and the corresponding constant amplitude fatigue limit (CAFL) for each AASHTO detail category is designated in the table below.

Detail Category	N_{min} (infinite fatigue life)	CAFL(ksi)
A	1.8 x 106 cycles	24
B	3.0 x 106 cycles	16
B'	3.5 x 106 cycles	12
C	4.4 x 106 cycles	10
C'	2.5 x 106 cycles	12
D	6.4 x 106 cycles	7.0
E	1.2 x 107 cycles	4.5
E'	2.2 x 107 cycles	2.6

6-02.3(13)C20 Finite Life Regime Testing

The number of cycles, N, to either failure or runout, associated with each of the ten data need not exceed N_{min}, designated in the table in Section 6-02.3(13)C19.

The detail category applicable to fatigue design shall be that corresponding to the highest of the AASHTO detail category fatigue resistance graphs representing a lower bound of all ten experimentally acquired data.

If all but one datum falls above a selected AASHTO S-N curve, that one datum may be discarded and replaced by three new data obtained through additional testing. The additional testing shall be conducted using the same stress range as that of the discarded datum. The three additional data shall be plotted along with the remaining nine data. The applicable detail category shall be that corresponding to the highest of the AASHTO detail category fatigue resistance graphs representing a lower bound of all twelve data, except as limited in the previous table. For any detail, only one datum may be discarded and subsequently replaced with three additional data for any set of ten original data.

The maximum fatigue resistance of any detail shall not exceed that associated with the fatigue category prescribed in the table below.

Type of Detail	Maximum Permitted Category
Welded Multiple Centerbeam-to-Support Bar Connections	C
Weld Stirrup Attachments for Single Support Bar Systems	B
Bolted Stirrup Attachments for Single Support Bar Systems	D
Groove Welded Centerbeam Splices[1]	C
Miscellaneous Welded Connections[2]	C
Miscellaneous Bolted Connections	D

[1]Groove welded full penetration splices may be increased to Category B if weld integrity is verified using non-destructive testing (NDT).

[2]Miscellaneous connections include attachments for equidistant devices.

The fatigue resistance for stirrups welded to a centerbeam flange shall not be taken greater than that defined using the fatigue details defined in Section 6.6 of the AASHTO LRFD Bridge Design Specifications. The applicable fatigue detail for the centerbeam flange and for the stirrup shall be either a “Longitudinally Loaded Groove-Welded Attachment” or a “Longitudinally Loaded Fillet-Welded Attachment”, depending upon the type of connection used.

6-02.3(13)C21 Infinite Life Regime Testing

The applicable constant amplitude fatigue limit (CAFL) for fatigue design may be selected as the highest CAFL of the AASHTO detail categories representing a lower bound to the experimentally acquired data. The CAFL of the AASHTO detail categories are designated in the table in Section 6-02.3(13)C19.

A unique CAFL (different from the CAFL categories delineated in Section 6.6 of the AASHTO LRFD Bridge Design Specifications) may be established if all ten data are within 4 ksi of that unique CAFL.

6-02.3(13)C22 Data Reporting for Fatigue Tests

Fatigue test results and observations shall be reported in the typical S-N format (logarithm (S) vs. logarithm (N)) with the log of the stress range plotted as the ordinate (y-axis). Additionally, the data shall be reported in tabular format. The table shall contain the following information:

1. Nominal stress range at the specific detail, $S_{r,eff}$.
2. Applied load range for each patch.
3. Number of cycles at initial observation of cracking (for reporting purposes only, not included as S-N data).
4. Number of cycles at failure or termination of the test, N, and the reason for stopping the test (failure or termination).
5. Type of crack as described in Section 6-02.3(13)C15. A detailed description of the fatigue crack shall be provided if the observed crack does not resemble any of the crack types described in Section 6-02.3(13)C15.

The following information shall also be reported:

1. Expansion joint system type and manufacturer.
2. Drawings depicting shape, size, and dimensions of the specimen.
3. Drawings depicting fixture details, including specimen orientation.
4. Section properties and dimensions of the centerbeam and support bar.
5. Centerbeam-to-support bar connection details:
    a. Weld procedure specifications for welded expansion joint systems.
    b. Bolt size, material specifications, location, and method of tightening for bolted expansion joint systems.

6-02.3(13)C23 Durability Testing of Elastomeric Support Bearings

This subsection provides guidelines for durability testing of the elastomeric support bearings typically used in modular expansion joint systems as specified In Sections 6-02.3(13)C24 and 6-02.3(13)C25. It is not applicable to compression springs, equidistant springs, or other elastomeric components.

Tests shall be performed dynamically on individual bearings. Fatigue life is evaluated by applying a displacement range to each specimen rather than a load or stress range.

Specimens shall comprise full scale bearing components representative of those installed in field applications. PTFE sliding surfaces or materials typically bonded to the elastomeric support bearings shall be fabricated as an integral part of the specimen.

Prior to testing, each specimen shall be visually inspected for flaws or defects that could plausibly affect fatigue resistance. All flaws or details shall be defined and recorded. Data obtained from specimens containing such anomalies shall not be excluded from the data set. Observed anomalies shall also be reported with the test data.

Test fixtures shall have the capability to adequately support and secure the specimen throughout the duration of the test. The fixture shall be designed and fabricated to such tolerances as required to assure that additional stresses will not be generated in the specimen as a consequence of fixture misalignment.

Loads shall be applied through hydraulic actuators or other similar loading devices. Fatigue testing shall be performed using displacement control. Displacement and load ranges shall be continuously monitored throughout the duration of the fatigue test to assure that desired displacement range and minimum preload are maintained.

Load shall be applied to the specimen through flat steel plates that are smooth and free of surface corrosion. These plates shall be sufficiently thick to assure even load distribution to the specimen.

6-02.3(13)C24 Dynamic Stiffness Test

Testing shall be conducted on each specimen to be subjected to fatigue testing in order to establish its dynamic stiffness for at least three different loading frequencies. The maximum of these loading frequencies shall be equal to the service load frequency corresponding to a vehicle traveling at 60 mph. The loading frequency, f, shall be calculated as:

$$f = 0.5 \cdot V / (g + b)$$

where

V	=	vehicle speed (60 mph at service load)
g	=	centerbeam gap (assume mid-range configuration)
b	=	centerbeam width

The load range applied during the dynamic stiffness test shall be that obtained from structural analysis using fatigue wheel load and wheel load distribution factors as specified in Sections 6-02.3(13)C24 and 6-02.3(13)C25.

Each dynamic stiffness test shall be performed three times. Data from individual tests shall be compared to assure consistency of test results.

6-02.3(13)C25 Bearing Fatigue Test

A minimum of three fatigue tests shall be required to establish the durability of each type of bearing.

The fatigue test shall be conducted using displacement control. The displacement (strain) range shall be applied using a sine or other smooth waveform at any frequency less than or equal to the service load frequency calculated in Section 6-02.3(13)C24. The magnitude of the applied displacement amplitude, Δ, shall be calculated as:

$$\Delta = R_v / K$$

where

R_v	=	vertical reaction force at the support bearing as obtained from structural analysis
K	=	dynamic stiffness of the support bearing as determined in Section 6-02.3(13)C24

A minimum precompression strain shall be maintained in the specimen throughout the duration of the test. This precompression strain shall be approximately equal to that present in a support bearing in a field installation. The magnitude of the applied cyclic strain shall be at least equal to the precompression strain.

The minimum and maximum dynamic load shall be recorded at the beginning of the test. The minimum and maximum dynamic load shall be monitored and periodically recorded throughout the duration of the test.

At the end of each applied displacement cycle, the displacement shall be held at the precompression level for no less than one half of the period of loading in order to facilitate heat dissipation. Artificial air flow devices (electrical fans) may be used to assist heat dissipation. Excessive heat generation will adversely affect the tested fatigue life.

A specimen shall be accepted as having passed the fatigue test criteria after withstanding 2 million cycles of loading without failure.

The following criteria shall constitute failure:

1. The elastomeric material exhibits excessive deterioration or cracking.
2. The measured minimum dynamic load falls to 30% of the initial dynamic load recorded at test initiation.
3. The measured dynamic load range decreases to half of the initial dynamic load range recorded at test initiation.

Data shall be reported in tabular format and shall contain the following information for each specimen tested:

1. Minimum (precompression) strain, maximum strain, displacement, and load at test initiation.
2. Type of loading impulse (sine wave, ramp, or other loading impulse types).
3. Number of cycles at initial observation of distress leading to failure (for reporting purposes only, not to be included in the data).
4. Number of cycles at failure.
5. A description of the mode of failure.

The following data shall also be reported for each specimen tested:

1. Bearing type and manufacturer.
2. Drawings depicting shape, size, and dimensions of the specimen including PTFE sliding surfaces or materials bonded to the specimen.
3. Drawings depicting fixture details, including specimen orientation.

6-02.3(13)C26 Fatigue Testing Laboratory

Fatigue testing shall be performed by an independent testing laboratory. Facilities known to be capable of performing fatigue testing as specified are identified in Section 6-02.3(13)C as supplemented in the Special Provisions.

6-02.3(13)C27 Fabrication

The expansion joint systems shall be fabricated consistent with the details, dimensions, material specifications, and procedures delineated in the shop plans. All fabrication procedures shall be in conformance with the Standard Specifications and the Special Provisions.

All expansion joint systems shall be fabricated by the same manufacturer.

Metallic attachments used to secure elastomeric seals to the centerbeams, if welded to the centerbeams and edge beams, shall be welded continuously along both their top and bottom edges.

All PTFE shall be bonded under controlled conditions and in strict accordance with written instructions provided by the PTFE manufacturer.

All PTFE surfaces shall be smooth and free of bubbles after completion of bonding operations.

All stainless steel sliding surfaces in contact with PTFE shall be polished to a Number 8 mirror finish.

Each stainless steel sheet shall be welded to the steel backing plate in accordance with current AWS specifications. The stainless steel sheet shall be clamped to provide full contact with the steel backing plate during welding. The welds shall not protrude above the sliding surface of the stainless steel sheet.

All steel surfaces, except those surfaces beneath stainless steel sheet, those to be bonded to PTFE, or those in direct contact with strip seals, shall be protected against corrosion by one of the following methods:

1. Zinc metallized in accordance with Section 6-07.3(14).
2. Hot-dip galvanized in accordance with AASHTO M 111.
3. Painted in accordance with Section 6-07.3(9). The color of the final coat, when dry, shall match the color chip SAE AMS Standard 595 Color No. 26357. The surfaces embedded in concrete shall be painted only with a shop coat of inorganic zinc silicate paint.

6-02.3(13)C28 Inspection

Each expansion joint system shall be subjected to and shall pass three levels of inspection in order to be accepted. These three levels are Quality Control Inspection, Quality Assurance Inspection, and Final Inspection. The manufacturer shall provide both Quality Control Inspection and Quality Assurance Inspection. The Contractor shall provide access to the Engineer for the Final Inspection.

Quality control inspection shall be provided by the manufacturer on a full time basis during the fabrication process of all major components to assure that the materials and Work meet or exceed the minimum requirements of the contract. Quality control inspection shall be performed by an entity having a line of responsibility distinctly different from that of the manufacturer's fabrication department.

Quality assurance inspection shall be performed by an independent inspection agency provided by the manufacturer. Quality assurance inspection is not required to be full time inspection but shall be performed during all phases of the manufacturing process.

Final inspection of each expansion joint system will be performed by the Engineer at the job site immediately prior to installation. The Contractor shall provide an accessible work area for this inspection. During final inspection, the Engineer will inspect each expansion joint system for proper alignment, complete bond between expansion joint strip seals and steel components, and proper steel stud placement.

There shall be no bends or kinks in the steel components, except as required to follow bridge deck grades and as specifically detailed on the shop plans. Straightening of unintended bends or kinks will not be permitted. All expansion joint systems exhibiting bends or kinks, other than those shown on the shop plans, shall be removed from the job site and replaced with a new expansion joint system at the expense of the Contractor. Expansion joint strip seals not fully bonded to the steel shall be fully bonded at no additional expense to the Contracting Agency.

Studs will be visually inspected and will be struck lightly with a hammer. Studs that do not have a complete end weld or emit tintinnabulation when struck lightly with a hammer shall be replaced. Studs located more than one inch, in any direction, from the location specified on the shop plans shall be carefully removed and a new stud shall be welded in the proper location. All stud replacements shall be at no additional expense to the Contracting Agency.

6-02.3(13)C29 Acceptance

Each expansion joint system shall pass all three levels of inspection specified in Section 6-02.3(13)C28 to qualify for acceptance. Expansion joint systems which fail any one of the three levels of inspection shall be replaced or repaired at no expense to the Contracting Agency and to the satisfaction of the Engineer. All proposed remedial procedures shall be submitted as Type 2E Working Drawings.

The Contractor shall ascertain that the manufacturer has met the fatigue resistance characterization and prequalification requirements of Sections 6-02.3(13)C1 and 6-02.3(13)C2 applicable to the specific expansion joint system being installed. The Contractor shall be responsible for all additional costs and/or time delays associated with selection of an alternative expansion joint system incurred as a result of noncompliance with these requirements, including the failure of the manufacturer to retest revised details or material substitutions of a previously prequalified system.

6-02.3(13)C30 Shipping and Handling

The expansion joint system shall be delivered to the job site and stored in accordance with the manufacturer's shop plans.

Lifting mechanisms, temperature adjustment devices, and temporary anchorages shall not be welded to the centerbeams or edge beams.

Damage to the expansion joint system during shipping or handling shall be just cause for rejection of the expansion joint system.

Damage to the corrosion protection system shall be repaired to the satisfaction of the Engineer.

6-02.3(13)C31 Pre-Installation Conference

A pre-installation conference shall be held 5 to 10-working days before the scheduled installation of the modular expansion joint assembly. The purpose of the conference shall be to discuss construction procedures, personnel, equipment to be used, methods to address congestion surrounding the assembly due to bridge deck steel reinforcing bars, expansion joint assembly supports and construction aids, and concrete placement and consolidation operations, including specific placement and consolidation surrounding the assembly support boxes. Those attending shall include, at a minimum, the superintendent, foremen in charge of erecting the joint assembly and placing the concrete encapsulating the assembly, and representatives from the modular expansion joint assembly manufacturer.

If the project includes more than one modular expansion joint assembly, and if the Contractor's key personnel change between installation operations, or at the request of the Engineer, additional conferences shall be held before each modular expansion joint assembly installation.

6-02.3(13)C32 Installation

A qualified installation technician shall be present at the job site to assure proper installation of each expansion joint system. This technician shall be a full time employee of the manufacturer of the specific expansion joint system being installed. The Contractor shall comply with all recommendations made by the expansion joint manufacturer's installation technician. Each expansion joint system manufacturer's installation technician shall certify to the Engineer that the manufacturer recommended installation procedures were followed. All certifications to the Engineer shall be in writing and shall be signed and dated by the manufacturer's installation technician.

Each expansion joint system shall be installed in strict accordance with the manufacturer's shop plans under Section 6-02.3(13)C2 and the recommendations of the manufacturer's installation technician. All centerbeam welded field splices shall be performed by a certified welder under the direct supervision of the manufacturer's qualified installation technician as specified above. The weld procedure shall have been submitted by the manufacturer and accepted in accordance with Section 6-02.3(13)C2. The welder shall have been trained and certified for performing those specific welds in accordance with the current AASHTO/AWS D1.5 Bridge Welding Code.

Each permanently installed expansion joint system shall match exactly the finished bridge deck profile and grades.

The Contractor shall exercise care at all times to protect each expansion joint system from damage. The Contractor shall protect concrete blockouts and supporting systems from damage and construction traffic prior to installation of the expansion joint systems. After installation, construction loads shall not be allowed on the expansion joint systems. The Contractor shall submit a Type 2 Working Drawing consisting of a proposed method of bridging over each expansion joint system to accommodate construction traffic.

Each expansion joint system shall be set to a gap width corresponding to the ambient temperature at the time of setting. This information is specified in the Plans and shall also be specified on the shop plans. All mechanical devices supplied by the joint system manufacturer, for the purpose of setting the expansion joint system to the proper gap width, will remain the property of the manufacturer. When no longer required, the devices shall be returned to the manufacturer.

All forms and debris that may impede movement of the expansion joint systems shall be removed.

Each expansion joint system shall be tested for watertightness after installation. The Contractor shall flood each completely installed expansion joint system with water to a minimum depth of three inches for a duration of at least one hour. If leakage is observed, the expansion joint system shall be repaired to the satisfaction of the Engineer at the Contractor's expense. The repair procedure shall be prepared by the expansion joint system manufacturer and shall be submitted as a Type 2 Working Drawing. After repairs are completed, the expansion joint shall be retested for leakage.

6-02.3(14) Finishing Concrete Surfaces

All concrete shall show a smooth, dense, uniform surface after the forms are removed. If it is porous, the Contractor shall bear the cost of repairing it. The Contractor shall clean and refinish stained or discolored surfaces.

Subsections A and B (below) describe two classes of surface finishing.

6-02.3(14)A Class 1 Surface Finish

The Contractor shall apply a Class 1 finish to all surfaces of concrete members to the limits designated in the Contract Plans.

The Contractor shall follow steps 1 through 8 below. When steel forms have been used and when the surface of filled holes matches the texture and color of the area around them, the Contractor may omit steps 3 through 8. To create a Class 1 surface, the Contractor shall:

1. Remove all bolts and all lips and edgings where form members have met;
2. Any holes or projections exceeding ¼ inch shall be repaired to provide a uniform finish. Holes shall be filled with mortar conforming to Section 9-20.4(2) at a 1:2 cement/aggregate ratio;

3. Thoroughly wash the surface of the concrete with water;
4. Brush on a mortar conforming to Section 9-20.4(2) at a 1:1 cement/aggregate ratio, working it well into the small air holes and other crevices in the face of the concrete;
5. Brush on no more mortar than can be finished in 1 day;
6. Rub the mortar off with burlap or a piece of carpet as soon as it takes initial set (before it reaches final set);
7. Fog-spray water over the finish as soon as the mortar paint has reached final set; and
8. Keep the surface damp for at least 2 days.

If the mortar becomes too hard to rub off as described in step 6, the Contractor shall remove it with a Carborundum stone and water. Random grinding is not permitted.

6-02.3(14)B Class 2 Surface Finish

The Contractor shall apply a Class 2 finish to all above-ground surfaces not receiving a Class 1 finish as specified above unless otherwise indicated in the Contract. Surfaces covered with fill do not require a surface finish.

To produce a Class 2 finish, the Contractor shall remove all bolts and all lips and edgings where form members have met and fill all form tie holes.

The Contractor shall remove all lifting embedments to 1 inch below the finished surface and fill the voids in accordance with Section 6-02.3(14)A, items two and three.

6-02.3(14)C Pigmented Sealer for Concrete Surfaces

The Contractor shall submit a Type 1 Working Drawing consisting of the pigmented sealer manufacturer's written instructions covering, at a minimum, the following:

1. Surface preparation.
2. Application methods.
3. Requirements for concrete curing prior to sealer application.
4. Temperature, humidity and precipitation limitations for application.
5. Rate of application and number of coats to apply.

All surfaces specified in the Plans to receive pigmented sealer shall receive a Class 2 surface finish (except that concrete barrier surfaces shall be finished in accordance with Section 6-02.3(11)A). The Contractor shall not apply pigmented sealer from a batch greater than 12 months past the initial date of color sample acceptance of that batch by the Engineer.

The pigmented sealer color or colors for specific concrete surfaces shall be as specified in the Special Provisions.

The final appearance shall be even and uniform without blotchiness, streaking or uneven color. Surface finishes deemed unacceptable by the Engineer shall be re-coated in accordance with the manufacturer's recommendations at no additional expense to the Contracting Agency.

For concrete surfaces such as columns, retaining walls, pier walls, abutments, concrete fascia panels, and noise barrier wall panels, the pigmented sealer shall extend to 1 foot below the finish ground line, unless otherwise shown in the Plans.

Pigmented Sealer Materials shall be a product listed in the current WSDOT Qualified Products List (QPL). If the pigmented sealer material is not listed in the current WSDOT QPL, a sample shall be submitted to the State Materials Laboratory in Tumwater for evaluation and acceptance in accordance with Section 9-08.3.

6-02.3(14)D Concrete Surface Finishes Produced by Form Liners

The concrete finishes listed in the table below shall be accomplished by the use of either a form liner selected from the products listed in the WSDOT Qualified Products List (QPL), or a form liner accepted by the State Bridge and Structures Architect and the Engineer. For acceptance of form liners not listed in the current WSDOT QPL, the Contractor shall submit Type 3 Working Drawings consisting of catalog cuts, other descriptive supporting information, and a 2-foot square physical sample of the form liner.

Concrete Finish	Horizontal joints in elastomeric form liners are permitted on surfaces greater than 8 feet in height[1] provided that the minimum form liner panel dimensions are:	
	Height (ft)	Width (ft)
Fractured Basalt Finish	8	2
Fractured Fin Finish	8	8
Fractured Granite Finish	8	8
Variable Depth Random Board Finish	8	8
¾ Inch Random Board Finish	8	8
Ribbed Finish	8	8
Striated Finish	8	8
Ashlar Stone Finish	8	8
Block Finish	8	8
Split Face Finish	8	6
River Rock Finish	4	8
Cascadian Stone Finish	4	8

[1]4 feet in height for River Rock Finish and Cascadian Stone Finish

Variable Depth Random Board Finish shall utilize an elastomeric form liner.

¾ Inch Random Board Finish shall utilize either an elastomeric or a plastic form liner. When specified in Contract documents to use wooden form liners, the concrete surface finish shall be achieved with reusable wooden form liners meeting the requirements of this Section and Section 6-02.3(14)D1.

For Cascadian Stone Finish, no partial rocks will be allowed in the finished pattern. The pattern shall be continuous across butt joints. Horizontal and vertical joints shall be adjusted as needed.

Form liners shall be placed with the pillars, fins, board lines and faux mortar/other joints normal to grade for barrier applications and vertical for all other applications.

Horizontal and vertical joints in ABS, plastic, or elastomeric form liners shall be spliced in accordance with the manufacturer's printed instructions. The Contractor shall submit a Type 1 Working Drawing consisting of the manufacturer's joint splice instructions.

Once the forms are removed, the Contractor shall treat the joint areas by patching or light sandblasting as required by the Engineer to ensure that the joints are not visible. The concrete formed with ABS and plastic form liners shall be given a light sandblast to remove the glossy finish.

Form liners shall be cleaned, reconditioned, and repaired before each use. Form liners with repairs, patches, or defects which, in the opinion of the Engineer, would result in adverse effects to the concrete finish shall not be used.

Care shall be taken to ensure uniformity of color throughout the textured surface. A change in form release agent will not be allowed.

All surfaces formed by the form liner shall also receive a Class 2 surface finish. Form ties shall be a type that leaves a clean hole when removed. All spalls and form tie holes shall be filled as specified for a Class 2 surface finish.

6-02.3(14)D1 ¾ Inch Random Board Finish Using Wooden Form Liners

The reusable wooden form liners shall conform to Section 6-02.3(17)J and the texture pattern shown in the Plans. The texture pattern shall be accomplished with ¾ inch thick battens in varying widths applied to the surface of the forms. The edge of all battens shall be sloped 15 degrees to facilitate form removal.

The Contractor shall submit a Type 3 Working Drawing consisting of a concrete panel test section, with the ¾ inch random board texture to be used and based on the pattern shown in the Plans. The test section shall be constructed using the forms and materials intended to construct the permanent structures. The test section shall be composed of two ten foot by ten foot form sections which shall be assembled to make a ten foot by 20 foot concrete surface section, and shall include the wall top treatment, and one horizontal joint treatment.

All cracks, holes, slits, gaps, and apertures in forms shall be plugged and caulked with molding plaster to remain completely watertight and withstand the pressures of concrete placement. Joints between the form units shall be sealed with silicone or latex caulking compound. Butt joints may be sealed with non-absorptive sponge tape. Construction joints and expansion joints shall be incorporated into the pattern of the face treatment.

Forms and form ties shall be designed to permit removal without damaging the finish. Prying against the face of the concrete will not be allowed.

Storage of formwork and form materials shall be in a manner to prevent damage or distortion. All damage to formwork during placing, removal, or storage shall be repaired by the Contractor at no additional expense to the Contracting Agency.

6-02.3(14)E Exposed Aggregate Finish

6-02.3(14)E1 Submittals

The Contractor shall submit Type 2 Working Drawings consisting of the following items:

1. Written description of the equipment to be used and procedure to be followed in producing the exposed aggregate finish.
2. A copy of the manufacturer's written instructions for applying the retardant coating and the clear sealer.
3. Type of nozzle, nozzle pressure, type and gradation of abrasive, blasting techniques, safety procedures, and containment methods and procedures used with all abrasive blasting and water blasting operations.
4. The method and materials used to collect, contain, and dispose of the concrete surface mortar removed from the finish surface, and the chemical agent residue and abrasives used to remove the concrete surface mortar.
5. For formed applications, a sample panel, equal either to the size of one concrete barrier panel minimum for barrier applications, or a four-foot by eight-foot panel for non-barrier applications, cast in a vertical position on the site and constructed in accordance with the procedure outlined in the Type 2 Working Drawing submittal.

6-02.3(14)E2 Producing Exposed Aggregate Finish

The Contractor shall produce all exposed aggregate concrete in accordance with procedure and equipment outlined in the Type 2 Working Drawing submittal. The exposed aggregate shall achieve the same final effect as demonstrated on the sample panel accepted by the Engineer.

Formwork shall be cleaned, reconditioned, and repaired before each use. Formwork with repairs, patches or defects which, in the opinion of the Engineer, would result in adverse effects to the concrete finish shall not be used.

Forms and form joints shall remain completely watertight. Butt joints and joints between form units used on surfaces which are to receive an exposed aggregate finish shall be tongue and grooved, or splined and shall be sealed with a caulking compound.

As an alternative to using tongue and grooved or splined joints, a closed cell polyvinylchloride foam sealer of 3/16 inch thickness with pressure-sensitive adhesive on one or both sides may be used to seal the butt joints between form units. The foam sealer shall be recessed by an amount such that when the form units are compressed to their final position, the foam sealer will be flush with the face of the form units. Adjacent formwork panels, if used, shall be in line and no offset shall occur between panels.

Forms for the exposed aggregate surface for members not yet supporting loads, including the members own load, may be removed as required to affect the exposed aggregate surface, provided the concrete has a minimum age of twelve hours and is of sufficient strength and hardness so as not to be damaged by the form removal operations and provided that curing and protection operations are maintained.

Removal of forms on the remaining concrete surfaces shall be in accordance with Section 6-02.3(17)N.

After the forms are stripped, the surface mortar shall be removed from the areas specified to receive the exposed aggregate finish.

The exposed aggregate finish shall be obtained by either one or a combination of the two methods described in Sections 6-02.3(14)E3 and 6-02.3(14)E4 as necessary to provide the specified exposed aggregate finish.

6-02.3(14)E3 Retardant Coating Method

A retardant coating conforming to Section 9-08.3(2)A shall be applied to the formwork where concrete surfaces with exposed aggregate finish are shown in the Plans.

For cast-in-place concrete the retardant shall have an effective life of not less than the length of time required for the Class EA concrete to be in place prior to the removal of forms plus 12 hours.

For slip-formed concrete barrier and horizontal to near-horizontal applications, the retardant shall have an effective life of not less than 24 hours. The Contractor shall remove the surface mortar two to three hours after applying the retardant coating.

Retardant shall be applied in accordance with the manufacturer’s instructions to remove the surface mortar.

The sealer and form release agent used on the form shall be compatible with the retardant and shall not react with the retardant to produce an undesirable effect on the exposed aggregate finish. The sealer and form release agent to be used on the form shall be as recommended by the manufacturer of the retardant.

Surface mortar shall be removed using one of the following methods:

1. Light abrasive blasting
2. Washing with water under pressure, avoiding excessive pressure which loosens individual aggregate particles.
3. A combination of both methods.

6-02.3(14)E4 Abrasive Blasting Method

As soon as forms are stripped, the exposed aggregate areas shall be abrasive blasted to remove the surface mortar. For slip-formed concrete barrier and horizontal to near-horizontal applications, this shall be done once the concrete has attained a minimum age of 12 hours and is of sufficient strength and hardness to prevent damage.

Adjacent materials and finishes shall be protected from dust, dirt and other damage during abrasive blasting operations. Corners and edge of patterns shall be carefully blasted using back-up boards to maintain a uniform corner or edge line.

The abrasive blast finishing shall be done in as continuous an operation as possible, utilizing the same work crew to maintain continuity of finish on each surface or area of work.

The type and gradation of abrasive grit used, the type of nozzle, nozzle pressure, and blasting techniques shall be as specified in the Type 2 Working Drawing submittal, and as required to expose the aggregate.

The Contractor shall be responsible for safety of the workers and shall equip each with air-fed helmets. The Contractor shall provide suitable enclosures for the collection of grit and dust from the abrasive blasting operation.

After receiving the Engineer's acceptance of the exposed aggregate finish, a 10 percent muriatic acid wash shall be applied to the exposed aggregate surfaces. Surfaces shall be flushed thoroughly with water following a 5 to 10 minute interaction period between the acid solution and the surface.

All stains and streaks on the exposed aggregate surface shall be removed before applying the clear sealer.

6-02.3(14)E5 Applying Clear Sealer

Two seal coatings of clear sealer conforming to Section 9-08.3(2)B shall be applied to the exposed aggregate surfaces in accordance with the manufacturer's recommended procedure.

6-02.3(14)E6 Containment

When producing exposed aggregate finish on concrete surfaces over water, the Contractor shall exercise care and use suitable means to collect and dispose of abrasives and chemical agents, and the resulting concrete surface mortar debris used in or resulting from the finishing of the exposed aggregate surfaces to prevent their entry into the environment surrounding the Structure.

6-02.3(14)F Permeon Treatment

The Contractor shall apply permeon treatment to all concrete surfaces specified in the Plans to receive permeon treatment. The Contractor shall use SAE AMS Standard 595 Color Number 30219 as the target color. The target color is intended as a reference for hue, and is not intended as a reference for opacity or luster. The Contractor is advised that this target color is based on the concentration formula and application rate identified in the QPL for each product. The concentration formula and application rate for products not listed in the QPL will be determined by the Engineer.

The permeon treatment shall be applied only by personnel approved by the manufacturer to apply the product. The Contractor shall furnish certificates of approval from the manufacturer, for the personnel scheduled to perform the work, to the Engineer prior to beginning the treatment operation.

The concrete shall be cured for the time period recommended by the manufacturer prior to receiving the permeon treatment coating.

The Contractor shall clean and prepare the concrete surfaces in accordance with the recommendations of the manufacturer for the use of the treatment product.

The Contractor shall apply the permeon treatment to the surfaces specified, in accordance with the recommendations of the manufacturer for the use of the treatment product.

The Contractor shall prevent permeon treatment from reaching surfaces not specified to receive the permeon treatment.

The Contractor shall prevent pigmented sealer from reaching surfaces that have received permeon treatment. Should pigmented sealer reach surfaces that have received permeon treatment, the pigmented sealer shall be removed and the permeon treatment repaired in accordance with Section 1-07.13.

6-02.3(15) Date Numerals

Standard date numerals shall be placed where shown in the Plans. The date shall be for the year in which the Structure is completed. When an existing Structure is widened or when traffic barrier is placed on an existing Structure, the date shall be for the year in which the original Structure was completed. Unit Contract prices shall cover all costs relating to these numerals.

6-02.3(16) Plans for Falsework and Formwork

The Contractor shall submit all plans for falsework and formwork as Type 2E Working Drawings. A submittal is not required for footing or retaining wall formwork if the concrete placement is 4 feet or less in height. Formwork plans are required for fixed form barriers regardless of height.

The design of falsework and formwork shall be based on:

1. Applied loads and conditions which are no less severe than those described in Section 6-02.3(17)A;
2. Allowable stresses and deflections which are no greater than those described in Section 6-02.3(17)B;
3. Special loads and requirements no less severe than those described in Section 6-02.3(17)C;
4. Conditions required by other Sections of 6-02.3(17).

The falsework and formwork plans shall be scale drawings showing the details of proposed construction, including: sizes and properties of all members and components; spacing of bents, posts, studs, wales, stringers, wedges and bracing; rates of concrete placement, placement sequence, direction of placement, and location of construction joints; identification of falsework devices and safe working loads as well as identification of all bolts or threaded rods used with the devices including their diameter, length, type, grade, and required torque. The falsework plans shall show the proximity of falsework to utilities or nearby Structures including underground Structures. Formwork accessories shall be identified according to Section 6-02.3(17)H. All assumptions, dimensions, material properties, and other data used in making the structural analysis shall be noted on the drawing.

The Contractor shall furnish associated design calculations to the Engineer as part of the submittal. The design calculations shall include the structural and geotechnical design of the foundation and shall show the stresses and deflections in all load-carrying members that are part of the falsework system. Construction details which may be shown in the form of sketches on the calculation sheets shall be shown in the falsework or formwork drawings as well. Falsework or formwork plans will not be accepted in cases where it is necessary to refer to the calculation sheets for information needed for complete understanding of the falsework and formwork plans or how to construct the falsework and formwork.

6-02.3(16)A Vacant

6-02.3(16)B Pre-Contract Review of Falsework and Formwork Plans

The Contractor may request pre-contract review of formwork plans for abutments, wingwalls, diaphragms, retaining walls, columns, girders and beams, box Structures, railings, and bulkheads. Plans for falsework supporting the bridge deck for interior spans between precast prestressed concrete girders may also be submitted for pre-contract review.

To obtain pre-contract review, the Contractor shall electronically submit drawings and design calculations in PDF format directly to: BridgeConstructionSupport@wsdot.wa.gov

The Bridge and Structures Office, Construction Support Engineer will return the falsework or formwork plan to the Contractor with review notes, an effective date of review, and revisions needed prior to use.

For each contract on which the pre-reviewed falsework or formwork plans will be used, the Contractor shall submit a copy to the Engineer. Construction shall not begin until the Engineer has given concurrence.

If the falsework or formwork being constructed has deviations to the preapproved falsework or formwork plan, the Contractor shall submit plan revisions for review and approval in accordance with Section 6-02.3(16).

6-02.3(17) Falsework and Formwork

Formwork and falsework are both structural systems. Formwork contains the lateral pressure exerted by concrete placed in the forms. Falsework supports the vertical and/or the horizontal loads of the formwork, reinforcing steel, concrete, and live loads during construction.

The Contractor shall set falsework, to produce in the finished Structure, the lines and grades indicated in the Contract Plans. The setting of falsework shall allow for shrinkage, settlement, falsework girder camber, and all structural camber the Plans or the Engineer require.

Concrete forms shall be mortar tight, true to the dimensions, lines, and grades of the Structure. Curved surfaces shown in the Contract Plans shall be constructed as curved surfaces and not chorded, except as allowed in Section 6-02.3(17)J. Concrete formwork shall be of sufficient strength and stiffness to prevent overstress and excess deflection as defined in Section 6-02.3(17)B. The rate of depositing concrete in the forms shall not exceed the placement rate in the formwork plan Working Drawing. The interior form shape and dimensions shall also ensure that the finished concrete will conform with the Contract Plans.

If the new Structure is near or part of an existing one, the Contractor shall not use the existing Structure to suspend or support falsework unless the Plans or Special Provisions state otherwise. For prestressed girder and T-beam bridge widenings or stage construction, the bridge deck and the diaphragm forms may be supported from the existing Structure or previous stage, if accepted by the Engineer. For steel plate girder bridge widenings or stage construction, only the bridge deck forms may be supported from the existing Structure or previous stage, if accepted by the Engineer. See Section 6-02.3(17)E for additional conditions.

On bridge decks, forms designed to stay in place made of steel or precast concrete panels shall not be used.

For post-tensioned Structures, both falsework and forms shall be designed to carry the additional loads caused by the post-tensioning operations. The Contractor shall construct supporting falsework in a way that leaves the Superstructure free to contract and lift off the falsework during post-tensioning. Forms that will remain inside box girders to support the placement of the bridge deck concrete shall, by design, resist girder contraction as little as possible. See Section 6-02.3(26) for additional conditions.

6-02.3(17)A Design Loads

The design load for falsework shall consist of the sum of dead and live vertical loads, and a design horizontal load. The minimum total design load for any falsework shall not be less than 100 lbs/sf for combined live and dead load regardless of Structure thickness.

The entire Superstructure cross-section, except traffic barrier, shall be considered to be placed at one time for purposes of determining support requirements and designing falsework girders for their stresses and deflections, except as follows:

> For concrete box girder bridges, the girder stem, diaphragms, crossbeam and connected bottom slabs may be considered self-supporting between the falsework bents at the time the top slab is place provided the following conditions are true:
>
> 1. Five or more days have elapsed since the stem walls were placed.
> 2. The distance between the falsework bents is less than or equal to four times the depth of the girder that has been placed in the preceding concrete placements.

Falsework bents shall be designed for the entire live load and dead load, including all load transfer that takes place during post-tensioning, and braced for the design horizontal load.

Dead loads shall include the weight of all successive placements of concrete, reinforcing steel, forms and falsework, and all load transfer that takes place during post-tensioning. The weight of concrete with reinforcing steel shall be assumed to be not less than 160 pounds per cubic foot.

Live loads shall consist of a minimum uniform load of not less than 25 psf, applied over the entire falsework plan area, plus the greater of:

1. Actual weights of the deck finishing equipment applied at the rails, or;
2. A minimum load of 75 pounds per linear foot applied at the edge of the bridge deck.

The design horizontal load to be resisted by the falsework bracing system in any direction shall be:

> The sum of all identifiable horizontal loads due to equipment, construction sequence, side-sway caused by geometry or eccentric loading conditions, or other causes, and an allowance for wind plus an additional allowance of 1 percent of the total dead load to provide for unexpected forces. In no case shall the design horizontal load be less than 3 percent of the total dead load.

The minimum horizontal load to be allowed for wind on each heavy-duty steel shoring tower having a vertical load carrying capacity exceeding 30 kips per leg shall be the sum of the products of the wind impact area, shape factor, and the applicable wind pressure value for each height zone. The wind impact area is the total projected area of all the elements in the tower face normal to the applied wind. The shape factor for heavy-duty steel shoring towers shall be taken as 2.2. Wind pressure values shall be determined from the following table:

Wind Pressure on Heavy-Duty Steel Shoring Towers

Height Zone (Feet Above Ground)	Wind Pressure Value	
	Adjacent to Traffic	At Other Locations
0 to 30	20 psf	15 psf
30 to 50	25 psf	20 psf
50 to 100	30 psf	25 psf
Over 100	35 psf	30 psf

The minimum horizontal load to be allowed for wind on all other types of falsework, including falsework girders and forms supported on heavy-duty steel shoring towers, shall be the sum of the products of the wind impact area and the applicable wind pressure value for each height zone. The wind impact area is the gross projected area of the falsework support system, falsework girders, forms and any unrestrained portion of the permanent Structure, excluding the areas between falsework posts or towers where diagonal bracing is not used. Wind pressure values shall be determined from the following table:

Wind Pressure on All Other Types of Falsework

Height Zone (Feet Above Ground)	Wind Pressure Value	
	For Members Over and Bents Adjacent to Traffic Openings	At Other Locations
0 to 30	2.0 Q psf	1.5 Q psf
30 to 50	2.5 Q psf	2.0 Q psf
50 to 100	3.0 Q psf	2.5 Q psf
Over 100	3.5 Q psf	3.0 Q psf

The value of Q in the above tabulation shall be determined as follows:

$$Q = 1 + 0.2W\text{; but Q shall not be more than 10.}$$

Where:

W = is the width of the falsework system, in feet, measured normal to the direction of the wind force being considered.

The falsework system shall also be designed so that it will be sufficiently stable to resist overturning prior to the placement of the concrete. The minimum factor of safety against falsework overturning in all directions from the assumed horizontal load for all stages of construction shall be 1.25. If the required resisting moment is less than 1.25 times the overturning moment, the difference shall be resisted by bracing, cable guys, or other means of external support.

Design of falsework shall include the vertical component (whether positive or negative) of bracing loads imposed by the design horizontal load. Design of falsework shall investigate the effects of any horizontal displacement due to stretch of the bracing. This is particularly important when using cable or rod bracing systems.

If the concrete is to be post-tensioned, the falsework shall be designed to support all increased or redistributed loads caused by the prestressing forces.

6-02.3(17)B Allowable Design Stresses and Deflections

The maximum allowable stresses listed in this section are based on the use of identifiable, undamaged, high-quality materials. Stresses shall be appropriately reduced if lesser quality materials are to be used.

These maximum allowable stresses include all adjustment factors, such as the short-term load duration factor. The maximum allowable stresses and deflections used in the design of the falsework and formwork shall be as follows:

6-02.3(17)B1 Deflection

Deflection resulting from dead load and concrete pressure for exposed visible surfaces shall not exceed $1/360$ of the span.

Deflection resulting from dead load and concrete pressure for unexposed non-visible surfaces, including the bottom of the deck slab between girders shall not exceed $1/270$ of the span.

In the foregoing, the span length shall be the center line to center line distance between supports for simple and continuous spans, and from the center line of support to the end of the member for cantilever spans. For plywood supported on members wider than 1½ inches, the span length shall be taken as the clear span plus 1½ inches. Also, dead load shall include the weight of all successive placements of concrete, reinforcing steel, forms and falsework self weight. Only the self weight of falsework girders may be excluded from the calculation of the above deflections provided that the falsework girder deflection is compensated for by the installation of camber strips.

Where successive placements of concrete are to act compositely in the completed Structure, deflection control becomes extremely critical. Maximum deflection of supporting members shall not exceed 1⁄500 of the span for members constructed in several successive placements (such as concrete box girder and concrete T-beam girder Structures). Falsework components shall be sized, positioned, and/or supported to minimize progressive increases in deflection of the Structure which would preload the concrete or reinforcing steel before it becomes fully composite.

6-02.3(17)B2 Timber

Each species and grade of timber/lumber used in constructing falsework and formwork shall be identified in the drawings. The allowable stresses and loads shall not exceed the lesser of stresses and loads given in the table below or factored stresses for designated species and grade in Table 7.3 of the *Timber Construction Manual*, latest edition, by the American Institute of Timber Construction.

Compression perpendicular to the grain reduced to 300 psi for use when moisture content is 19 percent or more (areas exposed to rain, concrete curing water, green lumber).	450 psi
Compression parallel to the grain but not to exceed 1,500 psi.	$\frac{480{,}000\text{ psi}}{(L/d)^2}$
Flexural stress for members with a nominal depth greater than 8 inches.	1,800 psi
Flexural stress psi for members with a nominal depth of 8 inches or less.	1,500 psi
The maximum horizontal shear.	140 psi
AXIAL tension.	1,200 psi
The maximum modulus of elasticity (E) for timber.	1,600,000 psi

Where:

L is the unsupported length; and

d is the least dimension of a square or rectangular column, or the width of a square of equivalent cross-sectional area for round columns.

The allowable stress for compression perpendicular to the grain, and for horizontal shear shall not be increased by any factors such as short duration loading. Additional requirements are found in other parts of Section 6-02.3(17). Criteria for the design of lumber and timber connections are found in Section 6-02.3(17)I.

Plywood for formwork shall be designed in accordance with the methods and stresses allowed in the *APA Design/Construction Guide for Concrete Forming* as published by the American Plywood Association, Tacoma, Washington. As concrete forming is a special application for plywood, wet stresses shall be used and then adjusted for forming conditions such as duration of load, and experience factors. Concrete pour pressures shall be in accordance with Section 6-02.3(17)J.

6-02.3(17)B3 Steel

For identified grades of steel, design stresses shall not exceed those specified in the *Steel Construction Manual*, latest edition, by the American Institute of Steel Construction, except as follows:

Compression, flexural but not to exceed $0.6F_y$	$\frac{12{,}000{,}000 \text{ psi}}{Ld/bt}$
The modulus of elasticity (E) shall be	29,000,000 psi

When the grade of steel cannot be positively identified as with salvaged steel and if rivets are present, design stresses shall not exceed the following:

Yield point f_y	30,000 psi
Tension, axial, and flexural	16,000 psi
Compression, axial except L/r shall not exceed 120	$14{,}150 - 0.37(KL/r)^2$ psi
Shear on gross section of the web of rolled shapes	9,500 psi
Web crippling for rolled shapes	22,500 psi
Compression, flexural but not to exceed 16,000 psi and L/b not greater than 39	$16{,}000 - 5.2(L/b)^2$ psi
The modulus of elasticity (E) shall be	29,000,000 psi

Where:

L is the unsupported length;

d is the least dimension of rectangular columns, or the width of a square of equivalent cross-sectional area for round columns, or the depth of beams;

b is the flange width;

t is the thickness of the compression flange;

r is the radius of gyration of the compression flange about the weak axis of the member; and

F_y is the specified minimum yield stress, psi, for the grade of steel used.

All dimensions are expressed in inches.

6-02.3(17)C Falsework and Formwork at Special Locations

In addition to the minimum requirements specified in Sections 6-02.3(17)A and 6-02.3(17)B, falsework towers or posts supporting beams directly over Roadways or railroads which are open to traffic or the public shall be designed and constructed so that the falsework will be stable if subjected to impact by vehicles. The use of damaged materials, unidentifiable material, salvaged steel or steel with burned holes or questionable weldments shall not be used for falsework described in this section. For the purposes of this Specification the following public or private facilities shall also be considered as "Roadways": pedestrian pathways and other Structures such as bridges, walls, and buildings.

The dimensions of the clear openings to be provided through the falsework for Roadways, railroads, or pedestrian pathways shall be as specified in the Contract.

Falsework posts or shoring tower systems which support members that cross over a Roadway or railroad shall be considered as adjacent to Roadways or railroads. Other falsework posts or shoring towers shall be considered as adjacent to Roadways or railroads only if the following conditions apply:

1. Located in the row of falsework posts or shoring towers nearest to the Roadway or railroad; and
2. Horizontal distance from the traffic side of the falsework to the edge of pavement is less than the total height of the falsework and forms; or
3. The total height of the falsework and forms is greater than the horizontal clear distance between the base of the falsework and a point 10 feet from the centerline of track.

The Contractor shall provide any additional features for the Work needed to ensure that the falsework will be stable for impact by vehicles; providing adequate safeguards, safety devices, protective equipment, and any other needed actions to protect property and the life, health, and safety of the public; and shall comply with the provisions in Sections 2-03.3 and 6-02.3(17)M. The falsework design at special locations, shall incorporate the minimum requirements detailed in this section, even if protected by concrete median barrier.

The vertical load used for the design of falsework posts and towers which support the portion of the falsework over openings, shall be the greater of the following:

1. 150 percent of the design load calculated in accordance with Section 6-02.3(17)B, but not including increased or redistributed loads caused by the post-tensioning forces; or
2. 100 percent of the design load plus the increased or redistributed loads caused by the post-tensioning forces.

Each falsework post or each shoring tower leg adjacent to Roadways or railroads shall consist of either steel with a minimum section modulus about each axis of 9.5 inches cubed or sound timbers with a minimum section modulus about each axis of 250 inches cubed.

Each falsework post or shoring tower leg adjacent to Roadways or railroads shall be mechanically connected to its supporting footing at its base, or otherwise laterally restrained, to withstand a force of not less than 2,000 pounds applied at the base of the post or tower leg in any direction except toward the Roadway or railroad track. Posts or tower legs shall be connected to the falsework cap and stringer by mechanical connections capable of resisting a load in any horizontal direction of not less than 1,000 pounds.

For falsework spans over Roadways and railroads, all falsework stringers shall be mechanically connected to the falsework cap or framing. The mechanical connections shall be capable of resisting a load in any direction, including uplift on the stringer, of not less than 500 pounds. All associated connections shall be installed before traffic is allowed to pass beneath the span.

When timber members are used to brace falsework bents which are located adjacent to Roadways or railroads, all connections shall be bolted through the members using ⅝-inch diameter or larger bolts.

Concrete traffic barrier shall be used to protect all falsework adjacent to traveled Roadways. The falsework shall be located so that falsework footings, mudsills, or piles are at least 2 feet clear of the traffic barrier and all other falsework members shall also be at least 2 feet clear of the traffic barrier. Traffic barrier used to protect falsework shall not be fastened, guyed, or blocked to falsework but shall be fastened to the pavement according to details shown in the Plans. The installation of concrete traffic barrier shall be completed before falsework erection is begun. The traffic barrier at the falsework shall not be removed until allowed by the Engineer. Falsework openings which are provided for the Contractor's own use (not for public use) shall also use concrete traffic barrier to protect the falsework, except the minimum clear distance between the barrier and falsework footings, mudsills, piles, or other falsework members shall be at least 3 inches.

Falsework bents within 20 feet of the center line of a railroad track shall be braced to resist the required horizontal load or 2,000 pounds whichever is greater.

Pedestrian openings through falsework shall be paved or surfaced with full width continuous wood walks which shall be wheelchair accessible and shall be kept clear. Pedestrians shall be protected from falling objects and water falling from construction above. Overhead protection for pedestrians shall extend at least 4 feet beyond the edge of the bridge deck. Plans and details of the overhead protection and pathway shall be submitted with the falsework Working Drawings. Pedestrian openings through falsework shall be illuminated by temporary lighting, constructed and maintained by the Contractor. The temporary lighting shall be constructed in accordance with local electrical code requirements. The temporary lighting shall be steady burning 60-watt, 120-volt lamps with molded waterproof lamp holders spaced at 25-foot centers maximum. All costs relating to pedestrian pathway paving, wood walks, overhead protection, maintenance, operating costs, and temporary pedestrian lighting shall be incidental to applicable adjacent items of Work.

6-02.3(17)D Falsework Support Systems: Foundations, Manufactured Shoring Towers, Caps, and Posts

Foundations for falsework shall be designed for conditions stated in this Section using methods shown in the AASHTO Standard Specifications for Highway Bridges Seventeenth Edition – 2002 for allowable stress design, the AASHTO *LRFD Bridge Design Specifications* for load and resistance factor design or the AASHTO *Guide Design Specifications for Bridge Temporary Works*. Allowable stresses for materials shall not exceed stresses and conditions allowed by Section 6-02.3(17)B.

6-02.3(17)D1 Vacant

6-02.3(17)D2 Vacant

6-02.3(17)D3 Bents, Shoring Towers, Piling, Posts, and Caps

Plans for falsework bents or shoring tower systems, including manufactured tower systems shall have plan, cross-section, and elevation view scale drawings showing all geometry. Show in the falsework plans the proximity of falsework to utilities and all other nearby Structures including underground Structures. The ground elevation, cross-slopes, relation of stringers to one another, and dimensions to posts or piling shall be shown in the falsework plans. Column, pile, or tower heights shall be indicated. Member sizes, wall thickness and diameter of steel pipe columns or piles shall be shown in the falsework plans. Location of wedges, minimum bearing area and type of wedge material shall be identified in the falsework plans. Bracing size, location, material and all connections shall be described in the falsework plans.

The relationship of the falsework bents or shoring tower systems to the permanent Structure's pier and footing shall be shown. Load paths shall be as direct as possible. Loads shall be applied through the shear centers of all members to avoid torsion and buckling conditions. Where loads cause twisting, biaxial bending, or axial loading with bending, the affected members shall be designed for combined stresses and stability.

Posts or columns shall be constructed plumb with tops and bottoms carefully cut to provide full end bearing. Caps shall be installed at all bents supported by posts or piling unless the falsework Working Drawings specifically permit otherwise. Caps shall be fastened to the piling or posts. The falsework shall be capable of supporting non uniform or localized loading without adverse effect. For example, the loading of cantilevered ends of stringers or caps shall not cause a condition of instability in the adjacent unloaded members.

Timber posts and piling shall be fastened to the caps and mudsills by through-bolted connections, drift pins, or other accepted connections. The minimum diameter of round timber posts shall be shown in the falsework plans. Timber caps and timber mudsills shall be checked for crushing from columns or piling under maximum load.

Steel posts and piling shall be welded or bolted to the caps and shall be bolted or welded to the foundation. Steel members shall be checked for buckling, web yielding, and web crippling.

Wedges shall be used to permit formwork to be taken up and released uniformly. Wedges shall be oak or close-grained Douglas fir. Cedar wedges or shims shall not be used anywhere in a falsework or forming system. Wedges shall be used at the top or bottom of shores, but not at both top and bottom. After the final adjustment of the shore elevation is complete, the wedges shall be fastened securely to the sill or cap beam. Only one set of wedges (with one optional block) shall be used at one location. Screw jacks (or other allowed devices) shall be used under arches to allow incremental release of the falsework.

Sand jacks may be used to support falsework and are used for falsework lowering only. Sand jacks shall be constructed of steel with snug fitting steel or concrete pistons. Sand jacks shall be filled with dry sand and the jack protected from moisture throughout its use. They shall be designed and installed in such a way to prevent the unintentional migration or loss of sand. All sand jacks shall be tested in accordance with Section 6-02.3(17)G.

When falsework is over or adjacent to Roadways or railroads, all details of the falsework system which contribute to the horizontal stability and resistance to impact shall be installed at the time each element of the falsework is erected and shall remain in place until the falsework is removed. For other requirements see Section 6-02.3(17)C.

Transverse construction joints in the Superstructure shall be supported by falsework at the joint location. The falsework shall be constructed in such a manner that subsequent pours will not produce additional stresses in the concrete already in place.

6-02.3(17)D4 Manufactured Shoring Tower Systems and Devices

Manufactured proprietary shoring tower systems shall be identified in the falsework plans by make and model and safe working load capacity per leg. The safe working load for shoring tower systems shall be based upon a minimum 2½ to 1 factor of safety.

The safe working load capacity, anticipated deflection (or settlement), make and model shall be identified in the falsework plans for manufactured devices such as: single shores, overhang brackets, support bracket and jack assemblies, friction collars and clamps, hangers, saddles, and sand jacks. The safe working load for shop manufactured devices shall be based on a minimum ultimate strength safety factor of 2 to 1. The safe working load for field fabricated devices and all single shores shall be based on a minimum ultimate strength safety factor of 3 to 1.

The safe working load of all devices shall not be exceeded. The design loads shall be as defined by Section 6-02.3(17)A. The maximum allowable free end deflection of deck overhang brackets under working loads applied shall not exceed $\frac{3}{16}$ inch measured at the edge of the concrete slab regardless of the fact that the deflection may be compensated for by pre-cambering or of setting the elevations high. The Contractor shall comply with all manufacturer's specifications; including those relating to bolt torque, placing washers under nuts and bolt heads, cleaning and oiling of parts, and the reuse of material. Devices which are deteriorated, bent, warped, or have poorly fitted connections or welds, shall not be installed.

Shoring tower or device capacity as shown in catalogs or brochures published by the manufacturer shall be considered as the maximum load which the shoring is able to safely support under ideal conditions. These maximum values shall be reduced for adverse loading conditions; such as horizontal loads, eccentricity due to unbalanced spans or placing sequence, and uneven foundation settlement.

Copies of catalog data and/or other technical data shall be furnished with the falsework plans to verify the load-carrying capacity, deflection, and manufacturers installation requirements of all manufactured products or devices proposed for use. Upon request by the Engineer, the Contractor shall furnish manufacturer certified test reports and results showing load capacity, deflection, test installation conditions, and identify associated components and hardware for shoring tower systems or other devices. In addition to manufacturer's requirements, the criteria shown in the following sections for manufactured proprietary shoring tower systems and devices shall be complied with when preparing falsework plans, calculations, and installing these shoring tower systems and devices as falsework.

Alternative criteria and/or systems shall be submitted as a Type 2 Working Drawing consisting of a written statement on the manufacturer's letterhead, signed by the shoring or device manufacturer (not signed by a material supplier or the Contractor) addressing the following:

1. Identity of the specific Contract on which the alternative criteria and/or system will apply;
2. Description of the alternative criteria and/or system;
3. Technical data and test reports;
4. The conditions under which the particular alternative criteria may be followed; and
5. That a design based on the alternative criteria will not overstress or over deflect shoring components or devices nor reduce the required safety factor.

In any case where the falsework drawings detail a manufactured product and the manufacturer's safe working load, load versus deflection curves, factor of safety, and installation requirements cannot be found in any catalog, the Engineer may require load testing in accordance with Section 6-02.3(17)G to verify the safe working load and deflection characteristics.

Tower leg loads shall not exceed the limiting values under any loading condition or sequence. All frame extensions and reduced capacity shall be shown in the falsework plans. Screw jacks shall fit tight in the leg assemblies without wobble. Screw jacks shall be plumb and straight. Shoring towers shall be installed plumb, and load distribution beams shall be arranged such that vertical loads are distributed to all legs for all successive concrete placements. There shall be no eccentric loads on shoring tower heads unless the heads have been designed for such loading. Shoring towers shall remain square or rectangular in plan view and shall not be skewed. There shall be no interchanging of parts from one manufactured shoring system to another. Bent or faulty components shall not be used.

For manufactured shoring towers that allow ganging of frames, the number of ganged frames shall be limited to one frame per opposing side of a tower, and the total number of legs per ganged tower shall not exceed eight legs. Ganged frames shall be installed in accordance with the manufacturer's published standards using the manufacturer's components. Other gang arrangements shall not be used.

For manufactured steel shoring tower systems, the Contractor shall have bracing designed and installed for horizontal loads and falsework overturning in accordance with Section 6-02.3(17)A. Minimum bracing criteria and allowable leg loads are described in the following paragraphs.

All shoring tower systems and bracing shall be thoroughly inspected by the Contractor for plumb vertical support members, secure connections, and straight bracing members immediately prior to, at intervals during, and immediately after every concrete placement. For manufactured shoring tower systems, the maximum allowable deviation from the vertical is ⅛ inch in 3 feet. If this tolerance is exceeded, concrete shall not be placed until adjustments have brought the shoring towers within the acceptable tolerance.

6-02.3(17)E Stringers, Beams, Joists, Bridge Deck Support, and Deck Overhangs

All stringers, beams, joists, and bridge deck support shall be designed for the design loads, deflections, and allowable stresses described in the preceding Section 6-02.3(17)A, B, and C and for the following conditions.

At points of support, stringers, beams, joists, and trusses shall be restrained against rotation about their longitudinal axis. The effect of biaxial bending shall be investigated in all cases where falsework beams are not set plumb and the Structure cross-slope exceeds 3 percent.

For box girder and T-beam bridges, the centerline of falsework beams or stringers shall be located within 2 feet of the bridge girder stems and preferably directly under the stems or webs. Stringers supporting formwork for concrete box girder and T-beam slab overhangs shall be stiff enough so that the differential deflection due to the placement of bridge deck concrete is no more than 3⁄16 inch between the outside edge of the bridge deck and the exterior web even if camber strips can compensate for the deflection.

Friction shall not be relied upon for lateral stability of beams or stringers. If the compression flange of a beam is not laterally restrained, the allowable bending stress shall be reduced to prevent flange buckling. If flange restraint is provided and since it is impossible to predict the direction in which a compression flange will buckle, positive restraint shall be provided in both directions. Flange restraint shall be designed for a minimum load of 2 percent of the calculated compression force in the beam flange at the point under consideration.

Camber strips shall be used to compensate for falsework take-up and deflection, vertical alignment, and the anticipated Structure dead load deflection shown in the camber diagram in the Contract Plans. Camber is the adjustment to the profile of a load-supporting beam or stringer so that the completed Structure will have the lines and grades shown in the Plans. The dead load camber diagram shown in the Contract Plans is the predicted Structure dead load deflection due to weight of applied concrete loads. This dead load camber shall be increased by:

1. Amount of anticipated falsework take up,
2. Anticipated deflection of the falsework beam or stringer under the actual load imposed, and
3. Vertical curve compensation, if any.

Camber strips shall be fastened by nailing to the top of wood members, or by clamping or banding in the case of steel members. Camber strips shall have sufficient contact bearing area to prevent crushing under total load. Camber strips are required when the total camber adjustment exceeds ¼ inch for exterior falsework stringers and ½ inch for interior stringers.

On concrete box girder Structures, the forms supporting the bridge deck shall rest on ledgers or similar supports and shall not be supported from the bottom slab except as provided below. The form supports shall be fastened within 18 inches of the top of the web walls, producing a clear span between web walls. The bridge deck forms may be supported or posted from the bottom slab if the following conditions are met:

1. Permanent access, shown in the Contract Plans, is provided to the cells, and the centerline to centerline distance between web walls is greater than 10 feet;
2. Falsework stringers designed for total load, stresses and deflections in accordance with Section 6-02.3(17)A and B are located directly below each row of posts;
3. Posts have adequate lateral restraint; and
4. All forms (including the bridge deck forms), posts, and bracing are completely removed.

The falsework and forms on concrete box girder Structures supporting a sloping web and deck overhang shall consist of a lateral support system which is designed to resist all rotational forces acting on the stem, including those caused by the placement of bridge deck concrete, bridge deck formwork mass, finishing machine, and other live loads. Stem reinforcing steel shall not be stressed by the construction of the bridge deck slab placement. Overhang brackets shall not be used for the support of bridge deck forms from sloping web concrete box girder bridges.

Deck slab forms between girders or webs shall be constructed such that there is no differential settlement relative to the girders. The support systems for form panels supporting concrete deck slabs and overhangs on girder bridges (such as steel plate girders and prestressed girders) shall be designed as falsework. Falsework supporting deck slabs and overhangs on girder bridges shall be supported directly by the girders so that there will be no differential settlement between the girders and the deck forms during placement of deck concrete.

6-02.3(17)F Bracing

All falsework bracing systems shall be designed to resist the horizontal design load in all directions with the falsework in either the loaded or unloaded condition. All bracing, connection details, specific locations of connections, and hardware used shall be shown in the falsework plans. Falsework diagonal bracing shall be thoroughly analyzed with particular attention given to the connections. The allowable stresses in the diagonal braces may be controlled by the joint strength or the compression stability of the diagonal. Timber bracing for timber falsework bents shall have connections designed in accordance with Section 6-02.3(17)I. All damaged cross-bracing, such as split timber members shall be replaced. Steel strapping shall avoid making sharp angles or right-angle bends. A means of preventing accidental loss of tension shall be provided for steel strapping. See Sections 6-02.3(17)A, B, and C for design loads and allowable stresses.

Bracing shall not be attached to concrete traffic barrier, guardrail posts, or guardrail.

To prevent falsework beam or stringer compression flange buckling, cross-bracing members and connections shall be designed to carry tension as well as compression. All components, connection details and specific locations shall be shown in the falsework plans. Bracing, blocking, struts, and ties required for positive lateral restraint of beam flanges shall be installed at right angles to the beam in plan view. If possible, bracing in adjacent bays shall be set in the same transverse plane. However, if because of skew or other considerations, it is necessary to offset the bracing in adjacent bays, the offset distance shall not exceed twice the depth of the beam.

All falsework and bracing shall be inspected by the Contractor for plumbness of vertical support members, secure connections, tight cables, and straight bracing members immediately prior to, during, and immediately after every concrete placement.

Bracing shall be provided to withstand all imposed loads during erection of the falsework and all phases of construction for falsework adjacent to a Roadway, sidewalk, or railroad track which is open to the public. All details of the falsework system which contribute to horizontal stability and resistance to impact, including the bolts in bracing, shall be installed at the time each element of the falsework is erected and shall remain in place until the falsework is removed. The falsework plans shall show provisions for supplemental bracing or methods to be used to conform to this requirement during each phase of erection and removal. Wind loads shall be included in the design of such bracing or methods. Loads, connections, and materials for falsework adjacent to Roadways, shall also be in accordance with Section 6-02.3(17)C.

6-02.3(17)F1 Cable or Tension Bracing Systems

When cables, wire rope, steel rod, or other types of tension bracing members are used as external bracing to resist horizontal forces, or as temporary bracing to support bents while falsework is being erected or removed adjacent to traffic, all elements of the bracing system shall be shown in the falsework plans. Bracing shall not be attached to concrete traffic barrier, guardrail posts, or guardrail. All damaged bracing, such as frayed and kinked guying systems shall be replaced. Wire rope shall avoid making sharp angles or right-angle bends and a means of preventing accidental loss of tension shall be provided. The following information shall be submitted as a Type 2 Working Drawing:

1. Cable diameter, rod, or tension member size, and allowable working load.
2. Location and method of attaching the cable, rod, or tension member to the falsework. The connecting device shall be designed to transfer both horizontal and vertical forces to the cable without overstressing the falsework component.
3. The type of cable connectors or fastening devices (such as wire rope clips, plate clamps, or other attachment devices) to be used and the efficiency factor for each type. If cables are to be spliced, the splicing method shall be shown.
4. Method of tightening cables, rods, or tension members after installation if tightening is necessary to ensure their effectiveness. Method of preventing accidental loosening.
5. Anchorage details, including the size and mass of concrete anchor blocks, the assumed coefficient of friction for surface anchorages, and the assumed lateral soil bearing capacity for buried anchorages.
6. Method of pre-stretching or preloading cable or tension members.
7. Determination of the potential stretch or elongation of the tension member under the design load and if the resulting lateral deflection will cause excessive secondary stresses in the falsework.

Copies of manufacturer’s catalog or brochure showing technical data pertaining to the type of cable to be used shall be furnished with the falsework plans. Technical data shall include the cable diameter, the number of strands and the number of wires per strand, ultimate breaking strength or recommended safe working strength, and any other information as may be needed to identify the cable.

In the absence of sufficient technical data to identify the cable, or if it is old and obviously worn, the Contractor shall perform cable breaking tests to establish the safe working load for each reel of cable furnished. For static guy cable the minimum factor of safety shall be 3 to 1. The Contractor shall provide the Engineer an opportunity to witness these tests.

When cable bracing is used to prevent the overturning of heavy-duty shoring, attention shall be given to the connections by which forces are transferred from the shoring to the cables. Cable restraint shall be designed to act through the cap system to prevent the inadvertent application of forces which the shoring is not designed to withstand. Cables shall not be attached to tower components.

Cable splices made by lapping and clipping with wire rope clip type clamps shall not be used. Other splicing methods may be used; however, at each location where the cable is spliced, cable strength shall be verified by a load test.

When cables are used as external bracing to resist overturning of a falsework system, the horizontal load to be carried by the cables shall be calculated as follows:

1. When used with heavy-duty shoring systems, cables shall be designed to resist the difference between 1.25 times the total overturning moment and the resistance to overturning provided by the individual falsework towers.

2. When used with pipe-frame shoring systems where supplemental bracing is required, cables shall be designed to resist the difference between 1.25 times the total overturning moment and the resistance to overturning provided by the shoring system as a whole.
3. When used as external bracing to prevent overturning of all other types of falsework, including temporary support during erection and removal of falsework at traffic openings, cables shall be designed to resist 1.25 times the total overturning moment.

The maximum allowable cable design load shall be determined using the following criteria:

1. If the cable is new, or is in uniformly good condition, and if it can be identified by reference to a manufacturer's catalog or other technical publication, the allowable load shall be the ultimate strength of the cable as specified by the manufacturer, multiplied by the efficiency of the cable connector, and divided by a safety factor of 3 (i.e., safe working load = breaking strength × connector efficiency/safety factor).
2. If the cable is used but still in serviceable condition, or is new or nearly new but cannot be found in a manufacturer's catalog, the Contractor shall perform load breaking tests. In this case, the cable design load shall not exceed the breaking strength, as determined by the load test, multiplied by the connector efficiency factor, and divided by a safety factor of 3.
3. If the cable is used and still in serviceable condition, or is a new or nearly new cable which cannot be identified, and if load breaking tests are not performed, the cable design load shall not exceed the safe working load shown in the wire rope capacities table multiplied by the cable connector efficiency.

   Cable connectors shall be designed in accordance with criteria shown in the following tables "Efficiency of Wire Rope Connections" and "Applying Wire Rope Clips". Cable safe working loads are provided in table "Wire Rope Capacities".

Efficiency of Wire Rope Connections (As compared to Safe Loads on Wire Rope)	
Type of Connection	**Connector Efficiency**
Wire Rope	100%
Sockets – Zink Type	100%
Wedge Sockets	70%
Wire Rope Clips – With Thimble	80%
Knot and Clip (Contractors Knot)	50%
Plate Clamp – 3 Bolt Type With Thimble	80%
Spliced Eye and Thimble:	
¼″ and smaller	100%
⅜″ to ¾″	95%
⅞″ to 1″	88%
1⅛″ to 1½″	82%
1⅝″ to 2″	75%
2⅛″ and larger	70%

Wire Rope Capacities Safe Load in Pounds for New Plow Steel Hoisting Rope 6 Strands of 19-Wires, Hemp Center (Safety Factor of 6)		
Diameter inches	Weight Lbs./Ft.	Safe Load Lbs.
¼	0.10	1,050
5⁄16	0.16	1,500
⅜	0.23	2,250
7⁄16	0.31	3,070
½	0.40	4,030
9⁄16	0.51	4,840
⅝	0.63	6,330
¾	0.95	7,930
⅞	1.29	10,730
1	1.60	15,000
1⅛	2.03	18,600
1¼	2.50	23,000
1⅜	3.03	25,900
1½	3.60	30,700
1⅝	4.23	35,700
1¾	4.90	41,300

6-02.3(17)F2 Applying Wire Rope Clips

The only correct method of attaching U-bolt wire rope clips to rope ends is to place the base (saddle) of the clip against the live end of the rope, while the “U” of the bolt presses against the dead end.

The clips are usually spaced about six rope diameters apart to give adequate holding power. A wire-rope thimble shall be used in the loop eye to prevent kinking when wire rope clips are used. The correct number of clips for safe application, and spacing distances, are shown below:

Number of Clips and Spacing for Safe Application			
Improved Plow Steel Rope Diameter inches	Number of Clips		Minimum
	Drop Forged	Other Material	Spacing (Inches)
⅜	2	3	3
½	3	4	3½
⅝	3	4	4
¾	4	5	4½
⅞	4	5	5¼
1	5	6	6
1⅛	6	6	6¾
1¼	6	7	7½
1⅜	7	7	8¼
1½	7	8	9

6-02.3(17)F3 Anchor Blocks

Concrete anchor blocks and connections used to resist forces from external bracing shall be shown in the falsework plans. Concrete anchor blocks shall be proportioned to resist both sliding and overturning. When designing anchor block stability, the weight of the anchor block shall be reduced by the vertical component of the cable or brace tension to obtain the net or effective mass to be used in the anchorage computations. The coefficient of friction assumed in the design shall not exceed the following:

	Friction Coefficient
Anchor block set on sand	0.40
Anchor block set on clay	0.50
Anchor block set on gravel	0.60
Anchor block set on pavement	0.60

Multiply the friction coefficient by 0.67 if it is likely the supporting material is wet or will become wet during the construction period.

The method of connecting the cable or brace to the anchor block is part of the anchor block design. The connection shall be designed to resist both horizontal and vertical forces.

6-02.3(17)F4 Temporary Bracing for Bridge Girders During Erection

Steel girders shall be braced in accordance with Section 6-03.3(7)A.

Prestressed concrete girders shall be braced sequentially during girder erection. The bracing shall be designed and detailed by the Contractor and shall be shown in the falsework/formwork Working Drawings. The Contractor shall furnish, install, and remove the bracing at no additional cost to the Contracting Agency.

At a minimum, the Contractor shall brace girders at each end and at midspan to prevent lateral movement or rotation. This bracing shall be placed prior to the release of each girder from the erection equipment. If the bridge is constructed with cast-in-place concrete diaphragms, the bracing may be removed once the concrete in the diaphragms has been placed and cured for a minimum of 24 hours.

6-02.3(17)F5 Temporary Bracing for Bridge Girders During Diaphragm and Bridge Deck Concrete Placement

Girders shall be braced to resist all temporary and construction loads, including those caused by the placing of precast concrete deck panels and concrete for the bridge deck. At a minimum, the Contractor shall brace concrete girders to prevent relative lateral movement and rotation at a spacing not to exceed 60 feet. The Contractor may consider the bracing effects of the diaphragms. The Contractor shall account for the added load from concrete finishing machines and other construction loadings in the design of the bracing.

Bracing shall be designed and detailed by the Contractor and shall be shown in the girder erection plan.

Falsework support brackets and braces shall not be welded to structural steel bridge members or to steel reinforcing bars.

These braces shall be furnished, installed, and removed by the Contractor at no additional expense to the Contracting Agency.

6-02.3(17)G Testing Falsework Devices

The Contractor shall establish the load capacity and deflection (or settlement) of all friction collars and clamps, brackets, hangers, saddles, sand jacks, and similar devices utilizing a recognized independent testing Laboratory accepted by the Engineer. Laboratory tests shall use the same materials and design that will be used on the project. Test loads shall be applied to the device in the same manner that the device will experience loading on the project. All bolts or threaded rods used with the device shall be identified as to diameter, length, type, grade, and torque. All wedges, blocks, or shims used with the device on the project shall also be tested with the device. All adjustable jack systems used as a part of a device shall be tested with the device and shall have its maximum safe working extended height identified. Devices shall not be tested in contact with the permanent Structure. Independent members with the same properties as the permanent Structure shall be used to test device connections.

At least 14 days prior to the test, the Contractor shall submit a Type 2 Working Drawing consisting of the test procedure and scale drawing showing how the device will be tested and how data will be collected. The Contractor shall provide the Engineer an opportunity to witness these tests.

The independent testing Laboratory shall provide a certified test report which shall be signed and dated. The test report shall clearly identify the device tested including trademarks and model numbers; identify all parts and materials used, including grade of steel, or lumber, member section dimensions; location, size, and the maximum tested extended height of all adjustable jacks; indicate condition of materials used in the device; indicate the size, length and location of all welds; indicate how much torque was used with all bolts and threaded rods. The report shall describe how the device was tested, report the results of the test, provide a scale drawing of the device showing the location(s) of where deflections or settlements were measured, and show where load was applied. Deflections or settlements shall be measured at load increments and the results shall be clearly graphed and labeled. Prior to installation of falsework devices named in this section, the Contractor shall submit Type 2 Working Drawings consisting of the certified test reports.

The safe working load for shop manufactured devices named in this section shall be derived by dividing the ultimate strength by a safety factor of 2.0. The safe working load for field fabricated or field modified devices (including the use of timber blocks or wedges with the device) shall be determined by dividing the ultimate strength by a safety factor of 3.0. Working load shall include masses of all successive concrete placements, falsework, forms, all load transfer that takes place during post-tensioning, and all live loads; such as workers, Roadway finishing machines, and concrete delivery systems. The maximum allowable free end deflection of deck overhang brackets with combined dead and live working loads applied shall be 3⁄16 inch even though deflection may be compensated for by pre-cambering or setting the elevations high. The Contractor shall comply with all manufacturer's specifications; including those relating to bolt torque, cleaning and oiling of parts, and the reuse of material. Devices which are deteriorated, bent, warped or have poorly fitted connections or welds, shall not be installed.

6-02.3(17)H Formwork Accessories

Formwork accessories such as form ties, form anchors, form hangers, anchoring inserts, and similar hardware shall be specifically identified in the formwork plans including the name and size of the hardware, manufacturer, safe working load, and factor of safety. The grade of steel shall also be indicated for threaded rods, coil rods, and similar hardware. Wire form ties shall not be used. Welding or clamping formwork accessories to Contract Plan reinforcing steel will not be allowed. Driven types of anchorages for fastening forms or form supports to concrete, and Contractor fabricated "J" hooks shall not be used. Field drilling of holes in prestressed girders is not allowed.

Taper ties may be used provided the following conditions are met:

1. The structure is not designed to resist water pressure (pontoons, floating dolphins, detention vaults, or other concrete systems that are either placed in water or hold water).
2. After the taper tie is removed, plugs designed and intended for plugging taper tie holes shall be installed at each face of concrete. The plug shall be installed a minimum of 1½ inches clear from the face of concrete.
3. After the plug is installed, the hole shall be cleaned of all grease, contamination and foreign matter.
4. Holes on the exposed faces of concrete shall be patched and finished to match the surrounding concrete.

The following table from ACI 347R-88 provides minimum safety factors for formwork accessories. The hardware proposed shall meet these minimum ultimate strength requirements or the manufacturer's minimum requirements, whichever provides the greater factor of safety. The Contractor shall attach copies of the manufacturer's catalog cuts and/or test data of hardware proposed to the formwork plans and submit the falsework and formwork Working Drawings with supporting calculations in accordance with Section 6-02.3(16). In situations where catalog cuts and/or test data are not available, testing shall be performed in accordance with Section 6-02.3(17)G.

Minimum Safety Factors of Formwork Accessories*		
Accessory	**Safety Factor**	**Type of Construction**
Form Tie	2.0	All applications.
Form Anchor	2.0	Formwork supporting form mass and concrete pressures only.
Form Anchor	3.0	Formwork supporting masses of forms, concrete, construction live loads, and impact.
Form Hangers	2.0	All applications.
Anchoring Inserts	2.0	Placed in previous opposing concrete placement to act as an anchor for form tie.

*Safety factors are based on ultimate strength of the formwork accessory.

The bearing area of external holding devices shall be adequate to prevent excessive bearing stress on form lumber. Form ties and form hangers shall be arranged symmetrically on the supporting members to minimize twisting or rotation of the members. Form tie elongation shall not exceed the allowable deflection of the wale or member that it supports. Inserts, bolts, coil rods, and other fasteners shall be analyzed and designed for appropriately combined bending, shear, torsion, and tension stresses. The formwork shall not be attached to Contract Plan rebar or rebar cages. However, the Contractor may install additional reinforcing steel for formwork anchorage.

Frictional resistance shall not be considered as contributing to the stability of a connection or connecting device, except those designed as friction connectors such as U-bolt friction-type connectors.

Form anchors and anchoring inserts shall be designed considering concrete strength at time of loading, available embedment, location in the member, and any other factors affecting their working strength, and shall be installed in concrete in accordance with the manufacturer's published requirements. Form anchors and anchoring inserts embedded in previous concrete placements shall not be loaded until the concrete has reached the required design strength. The required design strength of concrete for loading of an anchor shall be shown in the formwork drawing if it is assumed that the anchor will be loaded before the concrete has reached its 28-day strength.

Installation of permanent concrete inserts, such as form ties hangers, or embedded anchor assemblies, shall permit removal of all metal to at least ½ inch below the concrete surface. Holes shall be patched in accordance with Section 6-02.3(14). During removal of the outer unit, the bond between the concrete and the inner unit or rod shall not be broken.

6-02.3(17)I Timber Connections

Timber connections shall be designed in accordance with the methods, stresses, and loads allowed in the Timber Construction Manual, Third Edition by the American Institute of Timber Construction (AITC). Timber falsework and formwork connections shall be designed using wet condition stresses for all installations West of the Cascade Range crest line and by criteria provided in the following sections. Frictional resistance shall not be considered as contributing to the stability of a timber connection.

6-02.3(17)I1 Bolted Connections

Tabulated values in the AITC *Timber Construction Manual*, Current Edition are based on square posts. For a round post or pile, the main member thickness shall be the side of a square post having the same cross-sectional area as the round post used.

The AITC Table 6.20 for Douglas Fir-Larch bolt Group 3 and for Hem-Fir bolt Group 8 show design values for bolts to be used when the load is applied either parallel or perpendicular to the direction of the wood grain. When the load is applied at an angle to the grain, as is the case with falsework bracing, the design value for the main member shall be obtained from the Hankinson formula shown in the AITC manual.

Design values in the AITC Table 6.20 apply only to three-member joints (bolt in double-shear) in which the side members are each ½ the thickness of the main member. This joint configuration is not typical of bridge falsework where side members are usually much smaller than main members. For two-member joints (single shear bolt condition), the AITC Table 6.20 values shall be adjusted by a single shear load factor as follows:

1. 0.75 for installations East of the Cascade Range crest line, except as shown in item 3 below;
2. 0.50 for installations West of the Cascade Range crest line; and
3. 0.50 for load acting at an angle to the bolt axis, as is the case with longitudinal bracing when falsework bents are skewed.

Except for connections in falsework adjacent to or over railroads or Roadways, threaded rods and coil rods may be used in place of bolts of the same diameter with no reduction in the tabulated values. At openings for Roadways and railroads, all connections shall be bolted using ⅝-inch diameter or larger through bolts.

Bolt holes shall be a minimum 1/32 inch to a maximum ⅛ inch larger than the bolt diameter. A washer not less than a standard cut washer shall be installed between the wood and the bolt head and between the wood and the nut to distribute the bearing stress under the bolt head and nut and to avoid crushing the fibers. In lieu of standard cut washers, metal plates or straps with dimensions at least equal to that of a standard cut washer may be substituted.

When steel bars or shapes are used as diagonal bracing, the tabulated design values shown in AITC Table 6.20 for the main members loaded parallel to grain (P value) are increased 75 percent for joints made with bolts ½ inch or less in diameter, 25 percent for joints made with bolts 1½ inch in diameter, and proportionally for intermediate diameters. No increase in the tabulated values is allowed for perpendicular-to-grain loading (Q value).

Clearance requirements for end, edge, and bolt spacing distance shall be as shown below. All distances are measured from the end or side of the wood member to the center of the bolt hole. For members which are subject to load reversals the larger controlling distances shall be used for design. For parallel-to-grain loading, the minimum distances for full design load:

1. In tension, minimum end distance shall be seven times the bolt diameter;
2. In compression, minimum end distance shall be four times the bolt diameter; and
3. In tension or compression, the minimum edge distance shall be one and one-half times the bolt diameter.

For perpendicular-to-grain loading, the minimum distance for full design load:

1. Minimum end distance shall be four times the bolt diameter;
2. Edge distance toward which the load is acting shall be at least four times the bolt diameter; and
3. Distance on the opposite edge shall be at least 1½-bolt diameters.

Minimum clearance (spacing) between adjacent bolts in a row shall be four times the bolt diameter, measured center-to-center of the bolt holes.

When more than two bolts are used in a line parallel to the axis of the side member, additional requirements shall be followed as shown in the AITC manual.

6-02.3(17)I2 Lag Screw Connections

Design values for lag screws subject to withdrawal loading are found in AITC Table 6.27. Values for wood having a specific gravity of 0.51 for Douglas Fir-Larch or 0.42 for Hem-Fir shall be assumed when using the table. The withdrawal values are in pounds per inch of penetration of the threaded part of the lag screw into the side grain of the member holding the point, with the axis of the screw perpendicular to that member. The maximum load on a given screw shall not exceed the allowable tensile strength of the screw at the root section.

AITC recommends against subjecting lag screws to end-grain withdrawal loading. However, if this condition cannot be avoided, the design value shall be 75 percent of the corresponding value for withdrawal from the side grain.

Values in the Group II wood species column shall be used for Douglas Fir-Larch and the Group III wood species column shall be used for Hem-Fir. When the load is applied at an angle to the grain, as is the case with falsework bracing, the design value shall be obtained from the Hankinson formula shown in the AITC manual.

When lag screws are subjected to a combined lateral and withdrawal loading, as would be the case with longitudinal bracing when the falsework bents are skewed, the effect of the lateral and withdrawal forces shall be determined separately. The withdrawal component of the applied load shall not exceed the allowable value in withdrawal. The lateral component of the applied load shall not exceed the allowable lateral load value.

Lag screws shall be inserted in lead holes as follows:

1. The clearance hole for the shank shall have the same diameter as the shank, and the same depth of penetration as the length of unthreaded shank;
2. The lead hole for the threaded portion shall have a diameter equal to 60 to 75 percent of the shank diameter and a length equal to at least the length of the threaded portion. The larger percentile figure in each range shall apply to screws of the greater diameters used in Group II wood species;
3. The threaded portion of the screw shall be inserted in its lead hole by turning with a wrench, not by driving with a hammer; and
4. To facilitate insertion, soap or other lubricant shall be used on the screws or in the lead hole.

6-02.3(17)I3 Drift Pin and Drift Bolt Connections

When drift pins or drift bolts are used, the required length and penetration shall be determined using the following criteria. The lateral load-carrying capacity of drift pins and drift bolts driven into the side grain of a wood member shall be limited to 75 percent of the design values for a common bolt of the same diameter and length in the main member. For drift pin connections, the pin penetration into the connected members shall be increased to compensate for the absence of a bolt head and nut. For drift bolts or pins driven into the end grain of a member, the lateral load-carrying capacity shall be limited to 60 percent of the allowable side grain load (perpendicular to grain value) for an equal diameter bolt with nut. To develop this allowable load the drift bolt or pin shall penetrate at least 12 diameters into the end grain. To fully develop the allowable load of the drift bolts or pins, they shall be driven into predrilled holes, 1⁄16 inch less in diameter than the drift pin or bolt diameter.

The criteria shown in the AITC *Timber Construction Manual*, Current Edition shall apply to drift bolt or pin connection allowable loads for the following conditions:

1. Withdrawal resistance; and
2. When there are more than two drift bolts or pins in a joint, allowable loads shall be further reduced by applying applicable modification factors shown in the AITC Table 6.3.

6-02.3(17)I4 Nailed and Spiked Joints

Joints using nails or spikes shall conform to the provisions of AITC. For side grain withdrawal, the values in AITC Table 6.35 for wood having a specific gravity of 0.51 for Douglas Fir-Larch and a specific gravity of 0.42 for Hem-Fir shall be used. End grain withdrawal shall not be used. For lateral loading, the values in AITC Table 6.36 for wood species Group II for Douglas Fir-Larch and wood species Group III for Hem-Fir shall be used. Diameters listed in the tables apply to fasteners before application of protective coatings.

When more than one nail or spike is used in a joint, the total design value for the joint in withdrawal or lateral resistance shall be the sum of the design values for the individual nails or spikes.

The tabulated design values for lateral loads are valid only when the nail penetrates into the main member at least 11 diameters for Douglas Fir-Larch and 13 diameters for Hem-Fir. Note that the values are maximum values for the type and size of fastener shown. The tabulated values shall not be increased even if the actual penetration is exceeded.

When main member penetration is less than 11 diameters for Douglas Fir-Larch and 13 diameters for Hem-Fir, the design value shall be determined by straight-line interpolation between zero and the tabulated load, except that penetration shall not be less than ⅓ of that specified.

Double-headed or duplex nails used in falsework and formwork construction are shorter than common wire nails or box nails of the same size designation. They have less penetration into the main member and therefore their load-carrying capacity shall be adjusted accordingly.

Nail and spike minimum spacing in timber connections shall be as follows:

1. The average center-to-center distance between adjacent nails, measured in any direction, shall not be less than the required penetration into the main member for the size of nail being used; and
2. The minimum end distance in the side member, and the minimum edge distance in both the side member and the main member, shall not be less than ½ of the required penetration.

Allowable values for withdrawal and lateral load resistance are reduced when toe nails are used in accordance with the following:

1. For withdrawal loading, the design load shall not exceed ⅔ of the value shown in the applicable design table; and
2. For lateral loading, the design load shall not exceed ⅚ of the value shown in the applicable design table.

Toe nails are recommended to be driven at an approximate angle of 30 degrees with the piece and started approximately ⅓ of the length of the nail from the end or side of the piece.

6-02.3(17)I5 Timber Connection Adjustment for Duration of Load

Tabulated values for timber fasteners are for normal duration of load and may be increased for short duration loading, except for connections used in falsework and formwork for post tensioned Structures and staged construction sequences. Duration of load adjustment for timber connections shall not be allowed for all post tensioned Structures and for staged construction sequences where delayed and/or staged loading occurs for any type of concrete Structure. The adjustment for duration of load as described in this section applies only to design values for timber connectors, such as nails, bolts, and lag screws. Allowable stresses for timber and structural steel components used in the connection, as described in Section 6-02.3(17)B, are maximums and thus shall not be increased.

Tabulated values for nails, bolts, and lag screws may be adjusted by the following duration-of-load factors:

1. 1.25 for falsework design governed by the minimum design horizontal load or greater (3 percent or greater of the dead load),
2. 1.33 for falsework design governed by wind load, and
3. 2.00 for falsework design governed by impact loading.

6-02.3(17)J Face Lumber, Studs, Wales, and Metal Forms

Elements of this section shall be designed for the loads, allowable stresses, deflections, and conditions which pertain from other Subsections of Section 6-02.3(17).

Forms battered or inclined above the concrete will tend to lift up as concrete is placed and shall have positive anchorage or counterweights designed to resist uplift and shall be shown in the formwork plans. Where the concrete pouring sequence causes fresh concrete to be significantly higher along one side of tied forms than the opposite side, a positive form anchorage system shall be designed capable of resisting the imbalance of horizontal thrust, and prevent the dislocation and sliding of the entire form unit.

Wooden forms shall be faced with smooth sanded, exterior plywood. This plywood shall meet the requirements of the National Bureau of Standards, U.S. Product Standard PS 1, and the design specification of the American Plywood Association (APA). Each full sheet shall bear the APA stamp. The Contractor shall list in the form plans the grade and class of plywood. If the Engineer accepts the manufacturer’s certification of structural properties, the Contractor may use plywood that does not carry the APA stamp. Plywood panels stamped “shop” or “shop cutting”, shall not be used.

Plyform is an APA plywood specifically designed and manufactured for concrete forming. Plyform differs from conventional exterior plywood grades in strength and the exterior face panels are sanded smooth and factory oiled. Likewise, there is a significant difference between grades designated Class 1, Class 2, and Structural I Plyform.

The grades of plywood for various form applications shall be as follows:

1. **Traffic and Pedestrian Barriers** (except those that will receive an architectural surface treatment) – Plywood used for these surfaces shall be APA grade High-Density Overlaid (HDO) Plyform Class I. But if the Contractor coats the form to prevent it from leaving joint and grain marks on the surface, plywood that meets or exceeds APA grades B-B Plyform Class I or B-C (Group I species) may be used. Under this option, the Contractor shall provide for the Engineer's acceptance a 4-foot-square, test panel of concrete formed with the same plywood and coating as proposed in the form plans. This panel shall include one form joint along its centerline. The Contractor shall apply coating material, according to the manufacturer's instructions, before applying chemical release agents.
2. **Other Exposed Surfaces** (all but those on traffic and pedestrian barriers) – Plywood used to form these surfaces shall meet or exceed the requirements of APA grades B-B Plyform Class I or B-C (Group I series). If one face is less than B quality, the B (or better) face shall contact the concrete.
3. **Unexposed Surfaces** (such as the underside of the bridge deck between girders, the interiors of box girders, or other concrete exposed surfaces not viewable to the public, and traffic and pedestrian barriers where surfaces will receive an architectural treatment) – Plywood used to form these surfaces may be APA grade CDX, provided the Contractor complies with stress and deflection requirements stated elsewhere in these Specifications.

Form joints on an exposed surface shall be in a horizontal or vertical plane. But in wingwalls and box girders, side form joints shall be placed at right angles and parallel to the Roadway grade. Joints parallel to studs or joists shall be backed by a stud or joist. Joints at right angles to studs and joists shall be backed by a stud or other backing the Engineer accepts. Perpendicular backing is not required if studs or joists are spaced:

1. Nine inches or less on center and covered with ½-inch plywood, or
2. Twelve inches or less on center and covered with ¾-inch plywood.

The face grain of plywood shall run perpendicular to studs or joists unless shown otherwise on the Contractor's formwork Working Drawings. Proposals to deviate from the perpendicular orientation shall be accompanied by supporting calculations of the stresses and deflections.

Forming for all exposed curved surfaces shall follow the shape of the curve shown in the Contract Plans and shall not be chorded except as follows. On any retaining wall that follows a horizontal circular curve, the wall stems may be a series of short chords if:

1. The chords within the panel are the same length, unless otherwise allowed by the Engineer;
2. The chords do not vary from a true curve by more than ½ inch at any point; and
3. All panel points are on the true curve.

Where architectural treatment is required, the angle point for chords in wall stems shall fall at vertical rustication joints.

For exposed surfaces of abutments, wingwalls, piers, retaining walls, and columns, the Contractor shall build forms of plywood at least ¾ inch thick with studs no more than 12 inches on center. The Engineer may allow exceptions, but deflection of the plywood, studs, or wales shall never exceed $\frac{1}{360}$ of the span (or $\frac{1}{270}$ of the span for unexposed surfaces, including the bottom of the deck slab between girders).

All form plywood shall be at least ½ inch thick except on sharply curved surfaces. There, the Contractor may use ¼-inch plywood if it is backed firmly with heavier material.

Round columns or rounded pier shafts shall be formed with a self-supporting metal shell form or form tube that leaves a smooth, nonspiralling surface. Wood forms are not permitted.

Metal forms shall not be used elsewhere unless the Engineer is satisfied with the surface and allows use in writing. The Engineer may withdraw allowing use of metal forms at any time. If permitted to use a combination of wood and metal in forms, the Contractor shall coat the forms so that the texture produced by the wood matches that of the metal. Aluminum shall not be used for metal forms.

For design purposes, the Contractor shall assume that on vertical surfaces concrete exerts 150 pounds per square foot per foot of depth. However, when the depth is reached where the rate of placement controls the pressure, the following table applies:

Rate of Placing Feet per Hour	Pressure, Pounds per Square Foot for Temperature of Concrete as Shown	
	60°F	70°F and Above
2	470	375
3	640	565
4	725	625
5	815	690
6	900	750
7	990	815
8	1,075	875
9	1,165	935
10	1,250	1,000
15	1,670	1,300

The pressures in the above table have been increased to provide an allowance for the vibration and impact.

All corners shall be beveled ¾ inch. However, footings, footing pedestals, and seals need not be beveled unless required in the Plans.

All forms shall be as mortar-tight as possible with no water standing in them as the concrete is placed.

The Contractor shall apply a parting compound on forms for exposed concrete surfaces. This compound shall be a chemical release agent that permits the forms to separate cleanly from the concrete. The compound shall not penetrate or stain the surface and shall not attract dirt or other foreign matter. After the forms are removed, the concrete surface shall be dust-free and have a uniform appearance. The Contractor shall apply the compound at the manufacturer's recommended rate to produce a surface free of dusting action and yet provide easy removal of the forms.

The Engineer may reject forms that will not produce a satisfactory surface.

6-02.3(17)K Concrete Forms on Steel Spans

Concrete forms on all steel Structures shall be removable and shall not remain in place. Where needed, the forms shall have openings for truss or girder members. Each opening shall be large enough to leave at least 1½ inches between the concrete and steel on all sides of the steel member after the forms have been removed. Unit Contract prices cover all costs related to these openings.

The Contractor shall not weld any part of the form to steel members.

The compression member or bottom connection of cantilever formwork support brackets shall bear either within 6 inches maximum vertically of the bottom flange or within 6 inches maximum horizontally of a vertical web stiffener. The Contractor's bridge deck form system shall be designed to prevent rotation of the steel girder. This can be achieved by temporary struts and ties or other methods the Contractor shows to be effective. Partial depth cantilever formwork support brackets that do not conform to the above requirements shall not be used unless the Contractor submits Type 2E Working Drawings consisting of details showing the additional formwork struts and ties used to brace the steel girder against web distortion caused by the partial depth bracket.

If the Engineer permits bolt holes in the web to support form brackets, the holes shall be shop drilled unless otherwise allowed by the Engineer. The Contractor shall fill the holes with fully torqued ASTM F3125 Grade A325 bolts in accordance with Section 6-03.3(33). Each bolt head shall be placed on the exterior side of the web. There shall be no holes made in the flanges.

6-02.3(17)L Finishing Machine Support System

Before using finishing machines, the Contractor shall submit a Type 2 Working Drawing consisting of detailed drawings that show the system proposed to support it. The Contractor shall not attach this (or any other) equipment support system to the sides or suspend it from any girder unless the Engineer permits. The Engineer will not permit such a method if it will unduly alter stress patterns or create too much stress in the girder.

6-02.3(17)M Restricted Overhead Clearance Sign

6-02.3(17)M1 Vehicular Traffic

The Contractor shall notify the Engineer not less than 15 working days before the anticipated start of each falsework and girder erection operation whenever such falsework or girders will reduce clearances available to the public traffic. Falsework openings shall not be more restrictive to traffic than shown in the Contract Plans.

Where the height of vehicular openings through falsework is less than 15 feet, a W 12-2 “Low Clearance Symbol Sign” shall be erected on the Shoulder in advance of the falsework and two or more W 12-301 and/or W 12-302 signs shall be attached to the falsework to provide accurate usable clearance information over the entire falsework opening. The posted low clearance shall include an allowance for anticipated falsework girder deflection (rounded-up to the next whole inch) due to design dead load, including all successive concrete pours. W 12-302 signs shall be used to designate prominent clearance restrictions and limits of usable clearance. In addition, where the clearance is less than the legal height limit (14 feet), a W 12-2 sign shall be erected in advance of the nearest intersecting road or wide point in the road at which a vehicle can detour or turn around. A W 13-501 sign indicating the distance to the low clearance shall be installed below the advance sign. The Engineer will furnish the above noted signs and the Contractor shall erect and maintain them, all in accordance with Section 2-04.3(6).

6-02.3(17)M2 Railroad Traffic

When erecting falsework that restricts overhead clearance above a railroad track, the Contractor shall place restricted overhead clearance signs in accordance with the railroad requirements. Unit Contract prices cover all costs relating to these signs.

6-02.3(17)N Removal of Falsework and Forms

The Contractor may request to remove forms based on the criteria in the table below. Both compressive strength and minimum time criteria shall be met if both are listed in the applicable row. The minimum time shall be from the time of the last concrete placement in the forms. In no case shall the Contractor remove forms or falsework without the Engineer’s concurrence.

Concrete Placed In	Percent of Specified Minimum Compressive Strength[1]	Minimum Compressive Strength[1]	Minimum Time
Side forms not supporting the concrete weight, including columns, walls, crossbeams, nonsloping box girder webs, abutments, and traffic and pedestrian barriers.			3 days
		or	
		1,400 psi	18 hours
Side forms of footings, pile caps, and shaft caps.[2]			18 hours
Crossbeams, sloping box girder webs, struts, inclined columns, inclined walls, and other forms that support the concrete weight.	80		5 days
Bridge decks supported on stringers, beam, or girders.[3]	80		14 days
Box girders, T-beam girders, and flat-slab Superstructure.[3]	80		14 days
Arches[3]	80		21 days

[1]Strength shall be proved by test cylinders made from the last concrete placed into the form. The cylinders shall be cured according to FOP for AASHTO R 100.

[2]Curing compound shall be immediately applied to the sides when forms are removed.

[3]Where continuous spans or segments are involved, the time for all spans will be determined by the last concrete placed affecting any span.

Before releasing supports from beneath beams and girders, the Contractor shall remove forms from columns to enable the Engineer to inspect the column concrete.

Curing shall comply with the requirements of Section 6-02.3(11). The concrete surface shall not become dry during form removal if removed during the cure period.

Before placing forms for traffic and pedestrian barriers, the Contractor shall completely release all falsework under spans.

The Engineer may allow leaving in place forms for footings in cofferdams or cribs. This decision will be based on whether removing them would harm the cofferdam or crib and whether the forms will show in the finished Structure.

All cells of a box girder Structure which have permanent access shall have all forms completely removed, including the bridge deck forms. All debris and all projections into the cells shall be removed. Unless otherwise shown in the Plans, the bridge deck interior forms in all other cells where no permanent access is available, may be left in place.

Falsework and forms supporting sloping exterior webs shall not be released until the bridge deck and deck overhang concrete has obtained its removal strength and number of days criteria listed in the table above. Stem reshoring shall not be used.

Open joints shown in the Plans shall have all forms completely removed, including Styrofoam products and form anchors, allowing the completed Structure to move freely.

If the Contractor intends to support or suspend falsework and formwork from the bridge Structure while the falsework and formwork is being removed, the Contractor shall submit a Type 2 Working Drawing consisting of the falsework and formwork removal plan and calculations. The falsework and formwork removal plan shall include the following:

1. The location and size of all cast-in-place falsework lowering holes and how the holes are to be filled;
2. The location, capacity, and size of all attachments, beams, cables, and other hardware used to attach to the Structure or support the falsework and formwork;

3. The type, capacity and factor of safety, weight, and spacing of points of reaction of lowering equipment; and
4. The weight at each support point of the falsework and formwork being lowered.

All other forms shall be removed whether above or below the level of the ground or water. Sections 6-02.3(7) and 6-02.3(8) govern form removal for concrete exposed to sea water or to alkaline water or soil. The forms inside of hollow piers, girders, abutments, or other enclosed concrete surfaces shall be removed through openings shown in the Plans or as allowed by the Engineer.

6-02.3(17)O Early Concrete Test Cylinder Breaks

The fabrication, curing, and testing of the early cylinders shall be the responsibility of the Contractor. Early cylinders are defined as all cylinders tested in advance of the design age of 28 days whose purpose is to determine the in-place strength of concrete in a Structure prior to applying loads or stresses. The Contractor shall retain a testing Laboratory to perform this Work. Testing Laboratories' equipment shall be calibrated within 1 year prior to testing and testers shall be either ACI certified or qualified in accordance with AASHTO R 18.

The concrete cylinders shall be molded in accordance with FOP for AASHTO R 100 from concrete last placed in the forms and representative of the quality of concrete placed in that pour.

The cylinders shall be cured in the field in accordance with FOP for AASHTO R 100 Section 10.2 Field Curing.

The concrete cylinders shall be tested for compressive strength in accordance with AASHTO T 22. The number of early cylinder breaks shall be in accordance with the Contractor's need and as allowed by the Engineer.

The Contractor shall submit a Type 2 Working Drawing consisting of all test results, proof of equipment calibration, and tester's certification. The Contractor shall not remove forms without the concurrence of the Engineer.

All costs in connection with furnishing cylinder molds, fabrication, curing, and testing of early cylinders shall be included in the unit Contract prices for the various Bid items of Work involved.

6-02.3(18) Placing Anchor Bolts

The Contractor shall comply with the following requirements in setting anchor bolts in piers, abutments, or pedestals:

1. If set in the wet concrete, the bolts shall be accurately placed before the concrete is placed.
2. If the bolts are set in drilled holes and grouted, hole diameter shall exceed bolt diameter by at least 1 inch. Grouting shall comply with Section 6-02.3(20).
3. If the bolts are set in pipe and grouted, grouting shall comply with Section 6-02.3(20).
4. Resin bonded anchors systems shall comply with Section 6-02.3(18)A.
5. If freezing weather occurs before bolts can be grouted into sleeves or holes, they shall be filled with an accepted antifreeze solution (non-evaporating).

6-02.3(18)A Resin Bonded Anchors

The resin bonded anchor system shall include the nut, washer, and threaded anchor rod which is installed into hardened concrete with a resin bonding material.

Resin bonding material used in overhead and horizontal application shall be specifically recommended by the resin manufacturer for those applications. Resin bonding material used in submerged liquid environment shall be specifically recommended by the resin manufacturer for this application.

Threaded anchor rod and nuts, resin bonding material, embedment depth and ultimate anchor tensile capacity requirements are specified in Section 9-06.4.

The Contractor shall submit items 1 and 2 below to the Engineer for all resin bonded anchor systems. If the resin bonded anchor system and anchor diameter are not listed in the current WSDOT Qualified Products List, the Contractor shall submit items 1, 2 and 3 with the RAM in accordance with Section 1-06.1(2).

For resin bonded anchor systems that are installed in a submerged liquid environment the Contractor shall submit items 1, 2, and 4 below. If the resin bonded anchor system and anchor diameter are not listed in the current WSDOT Qualified Products List, the Contractor shall submit items 1, 2, 3, and 4 with the RAM in accordance with Section 1-06.1(2).

1. The resin manufacturer's written installation procedure for the anchors.
2. The manufacturer's certificate of compliance for the threaded anchor rod certifying that the anchor rod meets these requirements.
3. Test results by an independent laboratory certifying that the threaded anchor rod system meets the ultimate anchor tensile capacity specified in Section 9-06.4. The tests shall be performed in accordance with ASTM E 488.
4. For threaded anchors intended to be installed in submerged liquid environments the Contractor shall submit tests performed by an independent laboratory within the past 24 months which certifies that anchors installed in a submerged environment meet the ultimate anchor tensile capacity specified in Section 9-06.4.

The embedment depth of the anchors shall be as specified in the Plans. If the embedment depth of the anchor is not specified in the Plans then the minimum embedment depth shall not be less than the embedment depth specified in Section 9-06.4.

The anchors shall be installed in accordance with the resin manufacturer's written procedure.

Holes shall be drilled as specified in the Plans. Holes may be drilled with a rotary hammer drill when core drilling is not specified in the Plans. If holes are core drilled, the sides of the holes shall be roughened with a rotary hammer drill after core drilling.

Holes shall be prepared in accordance with the resin manufacturer's recommendations and shall meet the minimum requirements as specified herein. Holes drilled into concrete shall be thoroughly cleaned of debris, dust, and laitance prior to installing the threaded rod and resin bonding material. Holes shall not have standing liquid at the time of installation of the threaded anchor rod.

The anchor nuts shall be tightened to the following torques when the embedment equals or exceeds the minimum embedment specified.

Anchor Diameter (inch)	Minimum Torque (ft-lbs)	Maximum Torque (ft-lbs)
⅜	12	18
½	22	35
⅝	55	80
¾	106	140
⅞	165	190
1	195	225
1-¼	370	525

When the anchor embedment depth in the Plans is less than the minimum values specified in Section 9-06.4, the anchor nuts shall be tightened to the torque values specified in the Plans, or as recommended by the resin bonded anchor system manufacturer and approved by the Engineer.

6-02.3(19) Bridge Bearings

Bridge bearings include the following:

1. Elastomeric bearing pads conforming to Section 9-31.8(1).
   The Contractor shall adhere the elastomeric bearing pads to the concrete surface using the manufacturer's recommended adhesive product.
2. Fabricated bearing assemblies including, at a minimum, the following:
   a. High-Load Multi-Rotational (HLMR) bridge bearing assemblies, including the following:
      i. Disc bearings, functioning as guided unidirectional or fixed or multi-directional bearings, and consisting of an upper and a lower unit.
      ii. Spherical bearings, functioning as guided or fixed or multi-directional bearings, and consisting of an upper, a middle, and a lower unit.
   b. Fabric pad bearings and transverse stop bearings, functioning as guided bearings and consisting of an upper and a lower unit.
   c. Pin bearings, functioning as guided or fixed bearings, and consisting of an upper, a middle, and a lower unit.

The entire bearing assembly shall be supplied by a single bearing manufacturer.

6-02.3(19)A Design Requirements for HLMR Bearing Assemblies

The Contractor shall design HLMR bearing assemblies based on the AASHTO LRFD Bridge Design Specifications and the following:

1. The bearing assembly design requirements for loads, movements, and rotations shall be as shown In the Plans.
2. The bearing assembly shall be removable and replaceable by raising the bridge superstructure 1/4-inch maximum. The bearing shall be held in place by recessing the upper and lower keeper plates and by providing recessed bolted keeper bars on the side of bearing removal.
3. The area of the polyether urethane disc for disc bearings shall be designed for an unfactored stress of 5,000 psi ± 5 percent at full dead load and live load.
4. The mechanical interlock of the solid or woven PTFE sheets to the steel substrates shall be sufficient to develop a horizontal force equal to 10-percent of the maximum unfactored vertical load for spherical bearings with an external restrainer, and 25-percent of the maximum unfactored vertical load for spherical bearings without an external restrainer.
5. The area of the PTFE surface shall be designed so that the contact pressure does not exceed the maximum contact pressure specified in the AASHTO LRFD Bridge Design Specifications. The contact stress shall be determined at the strength limit state as specified In the AASHTO LRFD Bridge Design Specifications.
6. The minimum coefficient of friction on PTFE surfaces used for design shall be those corresponding to 68°F in the AASHTO LRFD Bridge Design Specifications.
7. The anchorage of the sole plates, masonry plates, and guide bars to the supporting structural element shall be designed for the maximum horizontal design force per bearing shown in the Plans, or 20-percent of the maximum unfactored vertical design force per bearing, whichever is greater.
8. The sole and masonry plates shall have leveling capabilities.
9. The guide bars shall maintain all guided components within the guides at all points of translation and rotation of the bearing.

6-02.3(19)B Submittals

6-02.3(19)B1 HLMR Bearing Design Calculations Submittal

The Contractor shall submit Type 2E Working Drawings consisting of design calculations for all HLMR bearing components, including the polyether urethane disc, shear pin, base plates, bearing plates, sole plates, masonry plates, guide bars, keeper plates and bars, and anchor bolts. The submittal shall include, at a minimum, the following:

1. Bending stresses in the plates due to bearing pressure at maximum design load and eccentricity.
2. Concrete bearing pressure under the plates at maximum bearing pressure and eccentricity.
3. Bearing clearances at maximum load and rotation. The calculated clearances shall include the effects of anticipated Initial set and modified center of rotation.
4. Shear stress in the disc bearing shear pin at maximum horizontal load.
5. Design of all connections and mating surfaces.
6. Compressive stress on all sliding surfaces at maximum and minimum design loads, including rotation.

6-02.3(19)B2 HLMR Bearing Manufacturer Experience Submittal

The Contractor shall submit a Type 1 Working Drawing consisting of the name of the HLMR bearing manufacturer with a certification of HLMR bearing manufacturing experience. The certification of experience shall include a list of at least five HLMR bearing Installations performed by the bearing manufacturer on previous projects. The list shall include the following Information at a minimum for each installation:

1. Project Name and Location (Bridge name and highway number).
2. Date of installation.
3. Governmental Agency/Owner.
4. Name, address and phone number of the Governmental Agency's/Owner's representative.

6-02.3(19)B3 Fabrication Shop Drawing Submittal

The Contractor shall submit Type 2 Working Drawings consisting of bearing fabrication shop drawings, including, at a minimum, the following:

1. Bearing schedule identifying location and bearing type as described in Section 6-02.3(19).
2. Minimum and maximum horizontal and vertical service loads for HLMR bearings.
3. Magnitude and direction of movements for HLMR bearings at all bearing support points.
4. Minimum and maximum rotation capacity for HLMR bearings.
5. Construction rotation requirements for HLMR bearings.
6. Plan and elevation of the assembled bearing and each of the components showing dimensions and tolerances.
7. Complete details of all components and sections showing all materials incorporated into the bearing.
8. All AASHTO, ASTM and other material designations.
9. All weld callouts with supporting Weld Procedure Specifications (WPSs) and associated Procedure Qualification Records (PQRs) as required.

10. All surface finishes and coating requirements.
11. Bearing manufacturer's recommendations and procedures for bearing assembly shipment, storage, and installation.

6-02.3(19)B4 Submittals of Acceptance Test Reports and Certificates

The Contractor shall submit the following production samples and test reports and certificates for fabricated bridge bearing assemblies as applicable:

1. A Type 2 Working Drawing consisting of a six-inch square by 1/8-inch thick sample of PTFE taken from the lot of production material.
2. A Type 2 Working Drawing consisting of a six-inch square by 1-inch thick sample of pre-formed fabric pad taken from the lot of production material.
3. Type 1 Working Drawings consisting of Manufacturers' Certificates of Compliance for the PTFE, polyether urethane, pre-formed fabric pad duck, silicone grease, epoxy gel, and resin filler.
4. Type 1 Working Drawings consisting of certified mill test reports for all steel and stainless steel in the bearing assemblies.
5. Type 1 Working Drawings consisting of certified test reports confirming that the pre-formed fabric pads meet the specific requirements of proof load.

6-02.3(19)B5 Quality Assurance and Final Shop Inspection Process Submittal

The Contractor shall submit a Type 1 Working Drawing consisting of the independent inspection entity performing the Quality Assurance Inspection and Final Shop Inspection as specified in Section 6-02.3(19)F. The submittal shall include, at a minimum, the name, address, phone number, and contact person of the inspection entity performing the Inspection, the proposed Quality Assurance Inspection Program, and the forms to be used for the Quality Assurance Inspection Program.

6-02.3(19)B6 HLMR Bearing Testing Procedure Submittal

The Contractor shall submit a Type 1 Working Drawing consisting of the name, address, phone number, and contact person of the testing entity performing the required bearing testing specified in Section 6-02.3(19)E.

The testing entity shall be one of the following:

1. An independent testing agency.
2. The HLMR bearing manufacturer, with Independent verification by the inspection entity performing the certified shop Inspection of the bearings.

6-02.3(19)B7 HLMR Bearing Assembly Inspection Reports and Certificates

The Contractor shall submit Type 1 Working Drawings consisting of the periodic inspection reports of the independent inspection entity performing the required certified shop inspection, at a frequency defined in the Quality Assurance Inspection Program of Sections 6-02.3(19)B5 and 6-02.3(19)F2. The daily inspection reports shall report on the shop fabrication and testing activities relating to the bearing assemblies, and their conformance to the specification requirements.

The Contractor shall submit written documentation from the bearing manufacturer and the independent inspection entity certifying that the bearing assemblies have been manufactured in full compliance with the specification requirements.

6-02.3(19)C Bearing Assembly Fabrication

The edges of all components shall be broken by grinding so that there are no sharp edges.

6-02.3(19)C1 Flatness and Manufacturing Tolerances

Flatness of bearing surfaces shall be determined by the following method:

1. A precision straightedge, longer than the nominal dimension to be measured shall be placed in contact with the surface to be measured as parallel to it as possible.
2. A feeler gauge having an accuracy equal to the tolerance allowed ± 0.001-inch, shall be selected and inserted under the straightedge.
3. Surfaces are acceptable for flatness if the feeler gauge does not pass under the straightedge.
4. In determining the flatness, the straightedge may be located in any position on the surface being measured.

Flatness tolerances are defined as follows:

1. Class A tolerance = 0.001 x nominal dimension
2. Class B tolerance = 0.002 x nominal dimension
3. Class C tolerance = 0.005 x nominal dimension

Manufacturing tolerances for bearing components are as follows:

Polyether Urethane Disc

Diameter: ± 1/8-inch

Thickness: - 0, +1/16-inch

Flatness: Class B tolerance

Discs shall be manufactured from a single piece

Spherically Curved Surfaces

Radii:	± 1-percent, surfaces shall be parallel to each other
Spherical Surface Profile:	± 0.0002Dh or 1/128-inches, whichever is greater, where D = length of chord (in inches) between the ends of the PTFE surface in the direction of rotation, and h = projection of the PTFE (in inches) above the top of the confining recess.

PTFE Sheet

Plan dimensions:	Total nominal design area -0, + 1/8-inch
Thickness:	- 0, +1/64-inch
Flatness:	Class A tolerance, both surfaces

Pre-formed Fabric Pad

Plan dimensions:	- 0, + 3/16-inch
Thickness:	- 1/16, + 3/16-inch
Surface Finish:	For pre-formed fabric pads fabricated from multiple layers, all pad edges shall be free from visible horizontal displacement between the individual layers

Stainless Steel Sheet

Plan dimension:	- 0, + 3/16-inch
Flatness:	Class A tolerance, both surfaces

Backing, Bearing, Masonry, Sliding, and Sole Plates of HLMR Bearings

Plan dimensions	
Greater than 30-inches:	- 0, + 3⁄16-inch
30-inches or less:	- 0, + 1⁄8-inch
Thickness:	- 1⁄32, + 1⁄8-inch
Flatness:	Class A tolerance, side in contact with steel, polyether urethane disc, or PTFE Class C tolerance, side in contact with epoxy gel or grout or concrete
Width and length of	
PFTE recess:	- 0, + 0.04-inch of PTFE sheet size

The maximum gap between the external restrainer and the circular base plate, and the walls of a recess and a recessed plate, for a spherical bearing shall be 0.04-inches.

Backing, Masonry, and Sole Plates of Pin, Fabric Pad, and Transverse Stop Bearings

Plan dimensions:	- 0, + 3⁄16-inch
Thickness:	- 0, + 3⁄16-inch
Flatness:	Class A tolerance, side in contact with stainless steel sheet, sole plate and preformed fabric pad Class C tolerance, side in contact with epoxy gel or grout or concrete
Width and length of	
PTFE recess:	- 0, + 1⁄16-inch of PTFE sheet size

The maximum gap between the external restrainer and the circular base plate, and the walls of a recess and a recessed plate, for a spherical bearing shall be 0.04-inches.

Guide Bar and Keeper Bar

Length:	± 1⁄8 inch
Section Dimensions:	± 1⁄16-inch
Flatness:	Class A tolerance, side in contact with steel or PTFE
Bar to bar tolerance:	± 1⁄32-inch

Bars shall not be more than 1⁄32-inch out of parallel

Bearing Block

Plan dimensions:	- 0, + 1⁄8 inch
Thickness:	± 0.015-inch
Groove radius for pin:	As shown in the Plans

Keeper ring grooves in bearing block

Radius, inner and outer:	± 0.005-inch
Depth of groove:	± 0.010-inch

Keeper Ring

Radius, inner and outer:	± 0.010-inch
Thickness:	± 0.030-inch

Pin

Length, shldr to shldr:	-0.020, + 0-inch
Diameter:	As shown in the Plans

Overall Height

HLMR bearing:	- 1/16, + 3/16-inch
Fabric pad bearing:	- 1/16, + 3/16-inch
Pin bearing:	- 0, + 10-percent

6-02.3(19)C2 HLMR Bearing Specific Fabrication Requirements

All bolted connections between structural steel surfaces shall meet the maximum spacing for sealing bolts in accordance with the AASHTO LRFD Bridge Design Specifications.

When the following components are shown in the Plans as part of the HLMR bearing assembly, the following specific fabrication requirements shall apply:

1. PTFE Sheet:
    a. The thickness of solid PTFE sheet shall be a minimum of 1/8-inch and a maximum of 3/16-inch. Solid PTFE sheet shall be recessed for a depth equal to one-half of its thickness into the material it is bonded to.
    b. The thickness of woven PTFE fabric, if used, shall be a minimum of 1/16-inch and a maximum of 1/8-inch.
    c. Dimpled PTFE, if shown in the Plans, shall be unfilled and shall have a maximum thickness of 3/16-inch. Dimples shall be placed on a ½-inch grid and have a depth of 1/16-inch.
    d. PTFE sheet shall be recessed and chemically bonded to the supporting steel plate or bar, except that woven PTFE sheet shall be mechanically bonded to the supporting plate or bar. Bonding shall be performed In accordance with the PTFE manufacturer’s written procedure.
    e. Following the bonding operation, the PTFE surface shall be smooth and free from bubbles. Filled PTFE shall be polished after the bonding operation is complete, in accordance with the AASHTO LRFD Bridge Construction Specifications.
2. Stainless Steel Sheet:
    a. The stainless steel sliding surface shall completely cover the PTFE surface in all operating positions plus one additional inch in all directions.
    b. The stainless steel shall be 14-gage thick for the main sliding surfaces and 10-gage thick for the guide bars.
    c. The stainless steel sheet shall be seal welded all around to the supporting steel plate or bar by the gas tungsten arc welding (GTAW) process in accordance with current AWS specifications. The stainless steel sheet shall be clamped down to have full contact with the supporting steel plate or bar during welding. The welds shall not protrude beyond the sliding surface of the stainless steel sheet.
    d. The curved surfaces of spherical bearings that receive stainless steel shall be weld overlaid to produce a surface chemistry equivalent to ASTM A240 Type 304L or 316L stainless steel as shown in the Plans.
    e. Stainless steel welded overlay on the curved surface of spherical bearings shall be a minimum of 3/32-inch thick after welding, grinding, and polishing.

3. Steel Plates and Bars:
   a. Sole plates and masonry plates shall be ¾-inch minimum thickness, unless otherwise shown in the Plans.
   b. Each guide bar and keeper bar shall be fabricated from a single steel plate.
   c. Guide bars and keeper bars shall be connected to the bearing assembly by recessing and bolting.
   d. The stainless steel sheet shall be welded to the guide bar or keeper bar before attaching the bar to the bearing assembly.
   e. The space between the guide bar or keeper bar and the guided component shall be 3⁄16-inch ± 1⁄16-inch.

6-02.3(19)C3 Non-HLMR Bearing Specific Fabrication Requirements

When the following components are shown in the Plans as part of the fabric pad bearing, pin bearing, or transverse stop bearing assembly, the following specific fabrication requirements shall apply:

1. PTFE Sheet:
   a. PTFE shall be ⅛-inch, unless otherwise shown in the Plans. PTFE shall be recessed for a depth equal to one-half of its thickness into the material it is bonded to, with the exposed height of PTFE not less than 3/64-inch.
   b. Dimpled PTFE, if shown in the Plans, shall be unfilled and shall have a maximum thickness of 3⁄16-inch. Dimples shall be placed on a 1/2-inch grid and have a depth of 1⁄16-inch.
   c. PTFE sheet shall be recessed and chemically bonded to the supporting steel plate or bar, except that woven PTFE sheet shall be mechanically bonded to the supporting plate or bar. Bonding shall be performed in accordance with the PTFE manufacturer's written procedure.
   d. Following the bonding operation, the PTFE surface shall be smooth and free from bubbles. Filled PTFE shall be polished after the bonding operation is complete, in accordance with the AASHTO LRFD Bridge Construction Specifications.
2. Stainless Steel Sheet:
   a. The stainless steel sheet shall be seal welded all around to the supporting steel plate or bar by the gas tungsten arc welding (GTAW) process in accordance with current AWS specifications.
   b. The stainless steel sheet shall be clamped down to have full contact with the supporting steel plate or bar during welding.
   c. The welds shall not protrude beyond the sliding surface of the stainless steel sheet.
3. Steel Plates and Bars:
   a. Each guide bar and keeper bar shall be fabricated from a single steel plate.
   b. Guide bars and keeper bars shall be connected to the bearing assembly by welding or bolting, as shown in the Plans.

6-02.3(19)D Corrosion Protection

Steel surfaces, except as otherwise specified below, shall be painted in accordance with Section 6-07.3(9), with a finish coat paint color as specified in Section 6-03.3(30) as supplemented in the Special Provisions. The surfaces of all welds fastening stainless steel to structural steel shall be painted as specified for structural steel. Stainless steel shall not be painted. Galvanized fastening hardware (anchor bolts, bolts, nuts, and washers) shall be painted in accordance with Section 6-07.3(11)A.

All coats of paint as specified in Section 6-07.3(9)A for steel surfaces shall be applied in the shop. After the bearing assembly has been erected in its final position with the anchor bolt nuts installed, all surfaces with damaged paint shall be repaired in accordance with Section 6-07.3(9)I.

All coats of paint as specified in Section 6-07.3(11)A for galvanized fastening hardware shall be applied after the bearing assembly has been erected in its final position with the anchor bolt nuts installed and tightened. The Contractor shall prepare the galvanized surfaces for painting in accordance with Section 6-07.3(11)A except only hand or power tool cleaning methods shall be used.

The embedded pipe assembly of the bearing assembly anchorage, when shown in the Plans, shall not be painted.

The following pin bearing components shall be painted only with one shop applied coat of inorganic zinc primer in accordance with Section 6-07.3(9).

1. Keeper rings.
2. Keeper ring groove surface in the bearing blocks.

The following pin bearing components and surfaces shall not be painted, but shall instead be coated with #2 extreme pressure grease:

1. Machined surfaces of the bearing blocks that contact the pin and keeper rings.
2. All surfaces of the pins.
3. All threads of the pin nuts.

The primer paint coated keeper rings shall be coated with #2 extreme pressure grease prior to final bearing assembly.

6-02.3(19)E HLMR Bearing Testing

The Contractor shall provide for HLMR bearing testing. The testing shall be performed by the testing entity selected in accordance with Section 6-02.3(19)B6.

All testing performed by the bearing manufacturer shall be witnessed by the inspection entity performing the certified shop inspection of the bearings.

Failure of the test bearing will result in rejection of all bearings.

The testing requirements specified below may be waived provided:

1. The bearing manufacturer, through the Contractor, shall submit a Type 1 Working Drawing consisting of certified test results from a previous installation of HLMR bearings of similar design and load capacity. This submittal shall accompany the design calculation submittal of Section 6-02.3(19)B1 and the fabrication shop plan submittal of Section 6-02.3(19)B3.
2. The tests performed on the previously installed bearings satisfy the requirements specified below.
3. All test requirements performed on and not satisfied by the previously installed bearings shall be performed on and satisfied by a test bearing in this Contract through a disc bearing Proof Load test conforming to Section 6-02.3(19)E1 or a spherical bearing Wear and Damage Characteristics test conforming to Section 6-02.3(19)E2, as appropriate.

The test bearing may be used as a production bearing provided:

1. The test bearing passed the test.
2. The test bearing was selected from the production bearings.
3. All PTFE in the test bearing assembly shall be replaced with new PTFE.

6-02.3(19)E1 Disc Bearing Proof Load Testing

When fabrication of disc bearings is complete, a Proof Load test shall be performed either on disc bearing assemblies randomly selected from the production bearings, or an equal number of prototype bearings with a minimum design capacity of 400-kips. One disc bearing per lot shall be tested where one lot is defined as a maximum of 25-production bearings.

The Proof Load test shall be performed on the selected test bearing assemblies as follows:

1. A proof load of 150-percent of the design capacity of the bearing shall be applied at the maximum design bearing rotation for a duration of five-minutes, removed, and then reapplied for five-minutes.
2. A bevel plate with a taper equal to the maximum design bearing rotation shall be used to simulate the specified bearing rotation.
3. After completing the specified load duration, the bearing shall be disassembled and inspected for wear and damage.
4. The test bearing shall show no signs of defects and failure while under load, and after disassembly and inspection.

6-02.3(19)E2 Spherical Bearing Wear and Damage Characteristics Testing

When fabrication of spherical bearings is complete, a Wear and Damage Characteristics test shall be performed on spherical bearing assemblies randomly selected from the production bearings. For bearings with a design capacity in excess of 1,000-kips, prototype bearings may be used for the Wear and Damage Characteristics test. One spherical bearing per lot shall be tested where one lot is defined as a maximum of 25-production bearings.

The Wear and Damage Characteristic test shall be performed on the selected test bearing assemblies as follows:

1. The bearing shall be subjected to 5,000-cycles of rotation (2.0 degrees each direction from level, 4.0 degrees total rotation) under the specified vertical dead load plus live load.
2. After completing the load cycles, the bearing shall be disassembled and inspected for wear and damage. A $\frac{1}{64}$-inch reduction in PTFE thickness, or damage to the bearing, shall be cause for rejection of the bearing assembly.
3. The test bearing shall show no signs of defects and failure while under load, and after disassembly and inspection.

6-02.3(19)F Bearing Inspection and Acceptance

Three levels of inspection shall be satisfied before the bearings are accepted. The manufacturer shall provide for both Quality Control and Quality Assurance Inspection in accordance with Section 6-02.3(19)F1 and 6-02.3(19)F2. The manufacturer shall provide access for the Final Shop Inspection in accordance with Section 6-02.3(19)F3.

The bearings shall satisfy each of the three levels of inspection as specified below prior to acceptance. Bearings that fail any one of the three levels of inspection shall have the deficiencies addressed in accordance with Section 1-05.7. All proposed corrective procedures shall be submitted as a Type 2 Working Drawing.

6-02.3(19)F1 Quality Control Inspection

During the fabrication process of all bearing assembly components and units, the manufacturer shall provide full time Quality Control Inspection to ensure that the materials and Work meet or exceed the minimum requirements of the Contract. Quality Control Inspection shall be the responsibility of the manufacturer's quality control group, which shall be independent of the fabrication group.

6-02.3(19)F2 Quality Assurance Inspection

Quality Assurance Inspection shall be performed by the independent inspection entity performing the certified shop inspection in accordance with Section 6-02.3(19)B5. Quality Assurance Inspection is not required to be full time inspection, but shall be done at all phases of the manufacturing process. The frequency of inspection shall be included in the Quality Assurance Inspection Program.

6-02.3(19)F3 Final Shop Inspection

Prior to shipping the bearings to the job site, a randomly selected representative number of production bearings shall be inspected by the independent inspection entity at the manufacturer's facility. The manufacturer shall provide a clean, dry, and enclosed area for the bearing inspection. The manufacturer shall disassemble and reassemble the bearings for inspection by the independent inspection entity. The independent inspection entity shall certify that the bearings have been inspected, and that the bearings have been manufactured in full compliance with the Contract requirements.

6-02.3(19)G Bearing Component Assembly, Shipping, and Storage

Each bearing, except bearing components welded to the bottom flange of steel girders or embedded into concrete superstructure, shall be fully assembled at the manufacturing plant and delivered to the construction site as a complete unit, ready for installation. The units shall be held together with removable restraints so that the sliding surfaces are not damaged. Softeners shall be placed under the restraints to protect all painted surfaces. The Contractor shall not damage the painted surfaces while shipping, storing and installing the bearing assemblies.

All bearing assemblies shall be marked with the following information prior to shipping:

1. Location of the bearing, including the pier and the specific location along the pier.
2. Direction arrow pointing in the ahead-on-station direction.

The above information shall be marked on the top plate of the upper unit of the bearing assembly. The marks shall be permanent and shall be visible after bearing installation.

The bearing assemblies shall have centerlines marked on both upper and lower units for checking alignment in the field.

The bearing assemblies shall be shipped in light-proof, moisture-proof and dust-proof containers.

6-02.3(19)H Bearing Assembly Field Inspection

The Contracting Agency may perform field inspection of bearing assemblies at the discretion of the Engineer. The Contractor shall provide a clean, dry and enclosed area at the site, spacious enough for the field inspection activities. The Contractor shall disassemble and reassemble the bearings for inspection by the Engineer. The disassembly and reassembly of the bearings shall be in accordance with the bearing manufacturer's written procedure and in the presence of the Engineer.

Bearings that fail the field inspection shall have the deficiencies addressed in accordance with Section 1-05.7. All proposed corrective procedures shall be submitted as a Type 2 Working Drawing.

6-02.3(19)I Bearing Assembly Installation

The Contractor shall install the bearing assembly in accordance with the installation procedure included with the fabrication shop drawing submittal required by Section 6-02.3(19)B3.

Sliding surfaces shall be finished true, lubricated, and installed level, or installed as shown in the Plans for transverse stop bearings.

PTFE sheet shall not be greased, except as otherwise noted. A thin uniform film of silicone grease shall be applied to the entire dimpled PTFE sheet before installation (all dimples shall be filled with grease).

For bearing assemblies with PTFE and stainless steel components, the Contractor shall take special care at all times to ensure protection of the PTFE and stainless steel surfaces from coming in contact with concrete and any other foreign matter.

The grout pad, and masonry plate when shown in the Plans, shall be formed and placed in accordance with Section 6-02.3(20), and installed level. The grout pad thickness shall be adjusted based on final bearing design dimensions, and to achieve final grade profile elevations as shown in the Plans. When shown with a masonry plate, the grout pad shall be pressure installed starting at the middle of the masonry plate.

For cast-in-place concrete superstructures, the upper units of bearing assemblies shall be anchored to the superstructure as shown in the Plans. For steel and precast concrete superstructures, the uppermost unit of bearing assemblies shall be connected or anchored to the superstructure as shown in the Plans.

When specified in the Plans for bearing assemblies supporting steel or precast concrete superstructure, the interface between the sole plate and the bridge superstructure (or the upper and lower sole plates when two separate components) shall be set with epoxy gel just before setting the superstructure in place. The (lower) sole plate surface in contact with the epoxy gel shall receive a thin uniform film of silicone grease, to prevent bonding to the epoxy gel. The threads of the sole plate clamping bolts shall be greased to prevent bonding and allow future removal. The Contractor shall apply the epoxy gel by troweling it onto the bottom surface of the steel girder flange or the upper sole plate welded to the steel girder flange and shall immediately bolt the (lower) sole plate in place to obtain a level surface.

Before the epoxy gel has cured, the steel or precast concrete superstructure shall be set in place, squeezing out the excess epoxy gel while filling the interface between the steel surfaces. Excess epoxy and grease shall be removed immediately. After the epoxy gel has cured, the sole plate clamping bolts shall be tightened to snug tight.

When the upper unit of the pin bearing consists of an upper bearing block welded to a sole plate, the top surface of the sole plate shall receive a thin uniform film of silicone grease, and the bolt threads connecting the pin assembly to the steel superstructure shall be greased, prior to fastening the sole plate to the steel superstructure.

Specified surfaces of the bearing blocks, pins, and pin nuts shall be coated with grease as specified in Section 6-02.3(19)D.

After installation, the orientation of the spherically curved units shall be ± ½ degree from level.

6-02.3(20) Grout for Anchor Bolts and Bridge Bearings

Grout shall conform to Section 9-20.3(2) for anchor bolts and for bearing assemblies with bearing plates. Grout shall conform to Section 9-20.3(3) for elastomeric bearing pads and fabric pad bearings without bearing plates.

Grout shall be a workable mix with a viscosity that is suitable for the intended application. Grout shall not be placed outside of the manufacturer recommended range of thickness. The Contractor shall receive concurrence from the Engineer before using the grout.

Field grout cubes and cylinders shall be fabricated and tested in accordance with Section 9-20.3 when requested by the Engineer, but not less than one per bridge pier or once per day.

Before placing grout, the substrate on which it is to be placed shall be prepared as recommended by the manufacturer to ensure proper bonding. The grout shall be cured as recommended by the manufacturer. The grout may be loaded when a minimum of 4,000 psi compressive strength is attained.

To grout bridge bearing masonry plates, the Contractor shall:

1. Build a form approximately 4 inches high with sides 4 inches outside the base of each masonry plate,
2. Fill each form to the top with grout,
3. Work grout under all parts of each masonry plate,
4. Remove each form after the grout has hardened,
5. Remove the grout outside each masonry plate to the base of the masonry plate,
6. Bevel off the grout neatly to the top of the masonry, and
7. Place no additional load on the masonry plate until the grout has set at least 72 hours.

After all grout under the masonry plate and in the anchor bolt cavities has attained a minimum strength of 4,000 psi, the anchor bolt nuts shall be tightened to snug tight. “Snug tight” means either the tightness reached by (1) a few blows from an impact wrench, or (2) the full effort of a person using a spud wrench. Once the nut is snug tight, the anchor bolt threads shall be burred just enough to prevent loosening of the nut.

6-02.3(21) Drainage of Box Girder Cells

To drain box girder cells, the Contractor shall provide and install, according to details in the Plans, short lengths of nonmetallic pipe in the bottom slab at the low point of each cell. The pipe shall have a minimum inside diameter of 4 inches. If the difference in Plan elevation is 2 inches or less, the Contractor shall install pipe in each end of the box girder cell. All drainage holes shall be screened in accordance with the Plan details.

6-02.3(22) Drainage of Substructure

The Contractor shall use weep holes and gravel backfill that complies with Section 9-03.12(2) to drain fill material behind retaining walls, abutments, tunnels, and wingwalls. To maintain thorough drainage, weep holes shall be placed as low as possible. Weep holes shall be covered with geotextile meeting the requirements of Section 9-33.2, Table 2 Class C before backfilling. Geotextile screening shall be bonded to the concrete with an accepted adhesive. Gravel backfill shall be placed and compacted as required in Section 3-07.3(1)E. In addition, if the Plans require, tiling, French or rock drains, or other drainage devices shall be installed.

If underdrains are not installed behind the wall or abutment, all backfill within 18 inches of weep holes shall comply with Section 9-03.12(4). Unless the Plans require otherwise, all other backfill behind the wall or abutment shall be gravel backfill for walls.

6-02.3(23) Opening to Traffic

Bridges with a bridge deck made of concrete shall remain closed to all traffic, including construction equipment, until the concrete has reached the 28-day specified compressive strength. This strength shall be determined with cylinders made of the same concrete as the bridge deck and cured under the same conditions. A concrete deck bridge shall never be opened to traffic earlier than 10 days after the deck concrete was placed and never before the Engineer allows.

For load restrictions on bridges under construction, refer to Section 6-01.6.

After curing bridge approach slabs in accordance with Section 6-02.3(11), the bridge approach slabs may be opened to traffic when a minimum compressive strength of 2,500 psi is achieved.

6-02.3(24) Reinforcement

Although a bar list is normally included in the Plans, the Contracting Agency does not guarantee its accuracy and it shall be used at the Contractor's risk. Reinforcement fabrication details shall be determined from the information provided in the Plans.

Before delivery of the reinforcing bars, the Contractor shall submit Type 1 Working Drawings consisting of an informational copy of the supplemental bending diagrams.

6-02.3(24)A Field Bending

Field bending of AASHTO M31 Grade 60 and ASTM A706 Grade 60 reinforcement shall be done in accordance with the requirements of this section. Field bending of all other reinforcement shall require a Type 2 Working Drawing showing the bend radii, bending and heating procedures, and all inspection or testing requirements.

Field bending shall not be done on reinforcement within the top or bottom third of column lengths or within plastic hinge regions identified in the Plans. Field bending shall not be done on bar sizes No. 14 or No. 18.

In field-bending steel reinforcing bars, the Contractor shall:

1. Make the bend gradually using a bending tool equipped with a bending diameter as listed in Table 1. Bending shall not be done by means of hammer blows and pipe sleeves. When bending to straighten a previously bent bar, move a hickey bar progressively around the bend.
2. Apply heat as described below for bending bar sizes No. 6 through No. 11 and for bending bar sizes No. 5 and smaller when the bars have been previously bent. Previously unbent bars of sizes No. 5 and smaller may be bent without heating when the bar temperature is 40°F or higher. When previously unbent bars of sizes No. 5 and smaller have a bar temperature lower than 40°F, they shall be heated to within the range of 100°F to 150°F prior to bending. In applying heat for field-bending steel reinforcing bars, the Contractor shall:
   a. Avoid damage to the concrete by insulating concrete within 6 inches of the heated bar area;
   b. Apply two heat tips simultaneously at opposite sides of bar sizes No. 7 or larger;
   c. Heat the bar to within the required temperature range shown in Table 2 as verified by using temperature-indicating crayons or other suitable means;
   d. Heat a minimum bar length as shown in Table 3. Locate the heated section of the bar to include the entire bending length;
   e. Bend immediately after the required temperature range has been achieved. Maintain the bar within the required temperature range during the entire bending process;
   f. Do not cool bars artificially with water, forced air, or other means.
3. Limit bends or straightening to these maximum angles: 135 degrees for bar sizes No. 8 or smaller, and 90 degrees for bar sizes No. 9 through No. 11.
4. Repair epoxy coating on epoxy coated bars in accordance with Section 6-02.3(24)H.

Table 1 Bending Diameters for Field-Bending Reinforcing Bars

	Bend Diameter/Bar Diameter Ratio	
Bar Size	Heat Not Applied	Heat Applied
No. 4, No. 5	8	8
No. 6 through No. 9	Not Permitted	8
No. 10, No. 11	Not Permitted	10

The minimum bending diameters for stirrups and ties for No. 4 and No. 5 bars when heat is not applied shall be specified in Section 9-07.

Table 2 Preheating Temperatures for Field-Bending Reinforcing Bars

Bar Size	Temperature (F)	
	Minimum	Maximum
No. 4	1,200	1,250
No. 5, No. 6	1,350	1,400
No. 7 through No. 9	1,400	1,450
No. 10, No. 11	1,450	1,500

Table 3 Minimum Bar Length to be Heated (d = nominal diameter of bar)

Bar Size	Bend Angle		
	45°	90°	135°
No. 4 through No. 8	8d	12d	15d
No. 9	8d	12d	Not Permitted
No. 10, No. 11	9d	14d	Not Permitted

6-02.3(24)B Protection of Materials

The Contractor shall protect reinforcing steel from all damage. When placed into the Structure, the steel shall be free from dirt, loose rust or mill scale, paint, oil, and other foreign matter.

When transporting, storing, or constructing in close proximity to bodies of salt water, plain and epoxy-coated steel reinforcing bar shall be kept in enclosures that provide protection from the elements.

If plain or epoxy-coated steel reinforcing bar is exposed to mist, spray, or fog that may contain salt, it shall be flushed with fresh water prior to concrete placement.

When the Engineer requires protection for reinforcing steel that will remain exposed for a length of time, the Contractor shall protect the reinforcing steel:

1. By cleaning and applying a coat of paint conforming to Section 9-08.1(2)B over all exposed surfaces of steel, or
2. By cleaning and painting paint conforming to Section 9-08.1(2)B on the first 6 inches of the steel bars protruding from the concrete and covering the bars with polyethylene sleeves.

The paint shall have a minimum dry film thickness of 1 mil.

Epoxy-coated steel reinforcing bars shall not be exposed to environmental conditions for a cumulative duration exceeding 60 days on site prior to full embedment in concrete. All provisions made to protect the reinforcing bars shall provide suitable protection from ultraviolet radiation including light and allow adequate ventilation to minimize condensation.

6-02.3(24)C Placing and Fastening

The Contractor shall position reinforcing steel as the Plans require and shall ensure that the steel is set within specified tolerances. Adjustments to reinforcing details outside of specified tolerances to avoid interferences and for other purposes are acceptable when approved by the Engineer.

When spacing between bars is 1 foot or more, they shall be tied at all intersections. When spacing is less than 1 foot, every other intersection shall be tied. If the Plans require bundled bars, they shall be tied together with wires at least every 6 feet. All epoxy-coated bars in the top mat of the bridge deck shall be tied at all intersections, however they may be tied at alternate intersections when spacing is less than 1 foot in each direction and they are supported by continuous supports meeting all other requirements of supports for epoxy-coated bars. Other epoxy-coated bars shall also be tied at all intersections, but shall be tied at alternate intersections when spacing is less than 1 foot in each direction. Wire used for tying epoxy-coated reinforcing steel shall be plastic coated. Tack welding is not permitted on reinforcing steel.

Abrupt bends in the steel are permitted only when one steel member bends around another. Vertical stirrups shall pass around main reinforcement or be firmly attached to it.

For slip-formed concrete, the reinforcing steel bars shall be tied at all intersections and cross braced to keep the cage from moving during concrete placement. Cross bracing shall be with additional reinforcing steel. Cross bracing shall be placed both longitudinally and transversely.

After reinforcing steel bars are placed in a traffic or pedestrian barrier and prior to slip-form concrete placement, the Contractor shall check clearances and reinforcing steel bar placement. This check shall be accomplished by using a template or by operating the slip-form machine over the entire length of the traffic or pedestrian barrier. All clearance and reinforcing steel bar placement deficiencies shall be corrected by the Contractor before slip-form concrete placement.

Precast concrete supports (or other accepted devices) shall be used to maintain the concrete coverage required by the Plans. The precast concrete supports shall:

1. Have a bearing surface measuring not greater than 2 inches in each dimension, and
2. Have a compressive strength equal to or greater than that of the concrete in which they are embedded.

In slabs, each precast concrete support shall have either: (1) a grooved top that will hold the reinforcing bar in place, or (2) an embedded wire that protrudes and is tied to the reinforcing steel. If this wire is used around epoxy-coated bars, it shall be coated with plastic.

Precast concrete supports may be accepted based on a Certificate of Compliance.

For precast concrete supports, the supplier's Certificate of Compliance or Contractor's Certificate of Compliance shall be on company letterhead, specifying the Contract Agency number, Contract title, the material being certified, the WSDOT Standards or Specifications being affirmed, and signed and dated by the company official.

In lieu of precast concrete supports, the Contractor may use metal or all-plastic supports to hold uncoated bars. Surfaces of metal chair supports that will not be covered by at least ½ inch of concrete shall be one of the following:

1. Hot-dip galvanized after fabrication in keeping with AASHTO M232 Class D;
2. Coated with plastic firmly bonded to the metal. This plastic shall be at least 3⁄32 inch thick where it touches the form and shall not react chemically with the concrete when tested in the State Materials Laboratory. The plastic shall not shatter or crack at or above -5°F and shall not deform enough to expose the metal at or below 200°F; or
3. Stainless steel that meet the requirements of ASTM A493, Type 302. Stainless steel chair supports are not required to be galvanized or plastic coated.

In lieu of precast concrete supports, epoxy-coated reinforcing bars may be supported by one of the following:

1. Metal supports coated entirely with a dielectric material such as epoxy or plastic,
2. Other epoxy-coated reinforcing bars, or
3. All-plastic supports.

Damaged coatings on metal bar supports shall be repaired prior to placing concrete.

All-plastic supports shall be lightweight, non-porous, and chemically inert in concrete. All-plastic supports shall have rounded seatings, shall not deform under load during normal temperatures, and shall not shatter or crack under impact loading in cold weather. All-plastic supports shall be placed at spacings greater than 1 foot along the bar and shall have at least 25 percent of their gross place area perforated to compensate for the difference in the coefficient of thermal expansion between plastic and concrete. The shape and configuration of all-plastic supports shall permit complete concrete consolidation in and around the support.

A "mat" is two adjacent and perpendicular layers of reinforcing steel. In bridge decks, top and bottom mats shall be supported adequately enough to hold both in their proper positions. If bar supports directly support, or are directly supported on No. 4 bars, they shall be spaced at not more than 3-foot intervals (or not more than 4-foot intervals for bars No. 5 and larger). Wire ties to girder stirrups shall not be considered as supports. To provide a rigid mat, the Contractor shall add other supports and tie wires to the top mat as needed.

Unless noted otherwise, the minimum concrete cover for main reinforcing bars shall be:

3 inches to a concrete surface deposited against earth without intervening forms.

2½ inches to the top surface of a concrete bridge deck or bridge approach slab.

2 inches to a concrete surface when not specified otherwise in this section or in the Contract documents.

1½ inches to a concrete barrier or curb surface.

Except for top cover in bridge decks and bridge approach slabs, minimum concrete cover to ties and stirrups may be reduced by ½ inch but shall not be less than 1 inch. Minimum concrete cover shall also be provided to the outermost part of mechanical splices and headed steel reinforcing bars.

Reinforcing steel bar location, concrete cover, and clearance shall not vary more than the following tolerances from what is specified in the Contract documents:

Reinforcing bar location for members 12 inches or less in thickness: ±0.25 inch

Reinforcing bar location for members greater than 12 inches in thickness: ±0.375 inch

Reinforcing bar location for bars placed at equal spacing within a plane: the greater of either ±1 inch or ±1 bar diameter within the plane. The total number of bars shall not be fewer than that specified.

The clearance between reinforcement shall not be less than the greater of the bar diameter or 1 inch for unbundled bars. For bundled bars, the clearance between bundles shall not be less than the greater of 1 inch or a bar diameter derived from the equivalent total area of all bars in the bundle.

Longitudinal location of bends and ends of bars: ±1 inch

Embedded length of bars and length of bar lap splices:

No 3 through No. 11	-1 in.
No. 14 through No. 18	-2 in.

Concrete cover measured perpendicular to concrete surface (except for the top surface of bridge decks, bridge approach slabs and other roadway surfaces): ±0.25 inch

Concrete cover measured perpendicular to concrete surface for the top surface of bridge decks, bridge approach slabs and other roadway surfaces: +0.25 inch, -0 inch

Before placing concrete, the Contractor shall:

1. Clean all mortar from reinforcement, and
2. Obtain the Engineer's permission to place concrete after the Engineer has inspected the placement of the reinforcing steel. (Concrete placed without the Engineer's permission will be rejected and the Contractor shall remove.)

6-02.3(24)D Splicing

The Contractor shall supply steel reinforcing bars in the full lengths the Plans require. Unless the Engineer concurs in writing, the Contractor shall not change the number, type, or location of splices.

The Engineer may permit the Contractor to use thermal or mechanical splices in place of the method shown in the Plans if they are of an accepted design. Use of a new design may be granted if:

1. The Contractor provides technical data and proof from the manufacturer that the design will perform satisfactorily, and
2. Sample splices and materials from the manufacturer pass the Engineer's tests.

The Contractor shall:

1. Not lap-splice reinforcing bars Nos. 14 or 18.
2. Not permit a welded or mechanical splice to deviate in alignment more than ¼ inch per 3½ feet of bar.
3. Distribute splices evenly, grouping them together only at points of low tensile stress.
4. Ensure at least 2 inches clearance between a splice and the nearest bar or the surface of the concrete (or 1½ inch for the length of the sleeve on mechanical splices).
5. Rigidly clamp or wire all splices in a way accepted by the Engineer.
6. Place lap-spliced bars in contact for the length of the splice and tie them together near each end.
7. Securely fasten the ends and edges of welded-wire-fabric reinforcement, overlapping them enough to maintain even strength.

6-02.3(24)D1 Splicing of Hoop Reinforcement for Columns and Shafts

When the Plans show steel reinforcement bar hoops, the hoops shall be spliced by one of the following methods:

1. Resistance butt weld splice, welded in accordance with Section 6-02.3(24)I.
2. Welded direct butt splice, welded in accordance with Section 6-02.3(24)J.
3. Welded lap splice if shown in the Plans, welded in accordance with Section 6-02.3(24)K.

All welded splices of hoop reinforcement shall be welded in the shop.

6-02.3(24)E Welding Reinforcing Steel

Welding of steel reinforcing bars shall conform to the requirements of ANSI/AWS D1.4 Structural Welding Code – Reinforcing Steel, latest edition, except where superseded by the Special Provisions, Plans, and these Specifications.

Before welding begins, the Contractor shall submit a Type 2 Working Drawing consisting of the welding procedure for each type of welded splice to be used, including the weld procedure specifications and joint details. The weld procedure specifications shall be written on a form taken from AWS D1.4 Annex A, or equivalent. Test results of tensile strength, macroetch, and visual examination shall be included. The form shall be signed and dated.

Welders shall be qualified in accordance with AWS D1.4. The Contractor shall be responsible for the testing and qualification of welders, and shall submit Type 2 Working Drawings consisting of welder qualification and retention records. The weld joint and welding position a welder is qualified in shall be in accordance with AWS D1.4. The welder qualifications shall remain in effect indefinitely unless, (1) the welder is not engaged in a given process of welding for which the welder is qualified for a period exceeding 6 months, or (2) there is some specific reason to question a welder's ability.

Filler metals used for welding reinforcing bars shall be in accordance with AWS D1.4 Table 5.1. All filler metals shall be low-hydrogen and handled in compliance with low-hydrogen practices specified in the AWS code.

Short circuiting transfer with gas metal arc welding will not be allowed. Slugging of welds will not be allowed.

For the purpose of compatibility with AWS D1.4, welded lap splices for spiral or hoop reinforcing shall be considered Flare-V groove welds, indirect butt joints.

The Contractor is responsible for using a welding sequence that will limit the alignment distortion of the bars due to the effects of welding. The maximum out-of-line permitted will be ¼ inch from a 3.5-foot straightedge centered on the weld and in line with the bar.

The ground wire from the welding machine shall be clamped to the bar being welded.

Where epoxy-coated steel reinforcing bars are specified to be spliced by welding, the epoxy coating shall be left off or removed from the surfaces to be heated, but in no cases less than six inches of each bar being welded. After the welding is complete, the Contractor shall apply epoxy patching material to the uncoated portions of the bar in accordance with Section 6-02.3(24)H.

6-02.3(24)F Mechanical Splices

The Contractor shall form mechanical splices with an Engineer-accepted system using sleeve filler metal, threaded coupling, or another method that complies with this section.

If necessary to maintain required clearances after the splices are in place, the Contractor shall adjust, relocate, or add stirrups, ties, and bars.

Before splicing, the Contractor shall provide the Engineer with the following information for each shipment of splice material:

1. The type or series identification (and heat treatment lot number for threaded-sleeve splices),
2. The grade and size of bars to be spliced,
3. A manufacturer's catalog with complete data on material and procedures,
4. A written statement from the manufacturer that the material is identical to that used earlier by the Engineer in testing and accepting the system design, and
5. A written statement from the Contractor that the system and materials will be used according to the manufacturer's instructions and all requirements of this section.

All splices shall meet these criteria:

1. Mechanical splices shall develop at least 125 percent of the specified yield strength of the unspliced bar. The ultimate tensile strength of the mechanical splice shall exceed that of the unspliced bar.
2. The total slip of the bar within the spliced sleeve of the connector after loading in tension to 30.0 ksi and relaxing to 3.0 ksi shall not exceed the following measured displacements between gage points clear of the splice sleeve:
   a. 0.01 inches for bar sizes up to No. 14.
   b. 0.03 inches for No. 18 bars.

3. The maximum allowable bar size for mechanical laps splices shall be No. 6.

The Engineer will visually inspect the splices and accept all that appear to conform with the test samples. For sleeve-filler splices, the Engineer will allow voids within the limits on file in the Working Drawing design submittal. If the Engineer considers a splice defective, it shall be removed and replaced at the Contractor's expense.

In preparing sleeve-filler metal splices, the Contractor shall:

1. Clean the bar surfaces by: (a) oxyacetylene torch followed by power wire brushing, or (b) abrasive blasting;
2. Remove all slag, mill scale, rust, and other foreign matter from all surfaces within and 2 inches beyond the sleeve;
3. Grind down projections on the bar that would prevent placing the sleeve;
4. Prepare the ends of the bars as the splice manufacturer recommends and as the accepted procedure requires; and
5. Preheat, just before adding the filler, the entire sleeve and bar ends to 300°F, plus or minus 50°F. (If a gas torch is used, the flame shall not be directed into the sleeve.)

When a metallic, sleeve-filler splice is used (or any other system requiring special equipment), both the system and the operator shall qualify in the following way under the supervision of the State Materials and Fabrication Inspector. The operator shall prepare six test splices (three vertical, three horizontal) using bars having the same AASHTO Designation and size (maximum) as those to be used in the Work. Each test sample shall be 6 foot plus the length of the splice. The bar alignment shall not deviate more than ⅛ inch from a straight line over the whole length of the sample. All six samples must meet the tensile strength and slip criteria specified in this section.

The Contractor shall provide labor, materials, and equipment for making these test samples at no expense to the Contracting Agency. The Contracting Agency will test the samples at no cost to the Contractor.

6-02.3(24)G Job Control Tests

As the Work progresses, the Engineer may require the Contractor to provide a sample splice (thermal or mechanical) to be used in a job control test. The operator shall create this sample on the job site with the Engineer present using bars of the same size as those being spliced in the Work. The sample shall comply with all requirements of these Specifications and is in addition to all other sample splices required for qualification.

The Engineer will require no more than two samples on a project with fewer than 200 splices and no more than one sample per 100 splices on a project with more than 200 splices.

6-02.3(24)H Epoxy-Coated Steel Reinforcing Bar

This Work is furnishing, fabricating, coating, and placing epoxy-coated steel reinforcing bars as the Plans, these Specifications, and the Special Provisions require. Coating material shall be applied electrostatically, by spraying, or by the fluidized-bed method.

All epoxy-coated bars shall comply with the requirements of Section 9-07. Fabrication may occur before or after coating.

The Contractor shall protect epoxy-coated bars from damage using padded or nonmetallic slings and straps free from dirt or grit. To prevent abrasion from bending or sagging, the Contractor shall lift bundled bars with a strong-back, multiple supports, or a platform bridge. Bundled bars shall not be dropped or dragged. During shop or field storage, bars shall rest on wooden or padded cribbing. The Contractor may substitute other methods for protecting the bars if the Engineer concurs. If the Engineer believes the coated bars have been badly damaged, they will be rejected.

Metal chairs and supports shall be coated with epoxy (or another inert coating accepted by the Engineer). The Contractor may use other support devices with the Engineer's concurrence. Plastic coated tie wires (accepted by the Engineer) shall be used to protect the coated bars from being damaged during placement.

The bars shall be placed as the Plans require and held firmly in place during placing and setting of the concrete. All bars shall be placed and fastened as specified in Section 6-02.3(24)C.

In the interval between installing coated bars and concreting the deck, the Contractor shall protect the coating from damage that might result from other construction Work.

The Engineer will inspect the coated bars after they are placed and before the deck concrete is placed. The Contractor shall patch all areas that show significant damage (as defined below).

Significant damage means an opening in the coating that exposes the steel in an area that exceeds:

1. 0.05 square inch (approximately ¼ inch square or ¼ inch in diameter or the equivalent).
2. 0.012 square inches (approximately ⅛ inch square or ⅛ inch in diameter) when the opening is within ¼ inch of another opening of equal or larger size.
3. 6 inches long, any width.
4. 0.50 square inch aggregate area in a 1 foot length of bar.

The Contractor shall patch significantly damaged areas with a patching material obtained from the epoxy resin manufacturer and accepted by the Engineer. This material shall be compatible with the coating and inert in concrete. Areas to be patched shall be clean and free of surface contaminants. Patching shall be done before oxidation occurs and according to the resin manufacturer's instructions.

6-02.3(24)I Resistance Butt Weld Splicing of Hoop Reinforcement for Columns and Shafts

6-02.3(24)I1 Splicing Quality Control Manager

The Contractor shall designate in writing a Splicing Quality Control Manager (SQCM). The SQCM shall be responsible for the quality of all hoop reinforcement splicing, including the inspection of materials and quality of Work, and submitting, receiving, and approving all correspondence, required submittals, and reports regarding hoop reinforcement splicing to and from the Engineer.

6-02.3(24)I2 Splice Sample Test Facilities

Qualification testing and testing of production sample splices shall be performed at an independent qualified testing laboratory at no additional expense to the Contracting Agency. The laboratory shall have the following:

1. Proper facilities, including a tensile testing machine capable of breaking full-size samples of all steel reinforcing bar splices.
2. Operators who have received documented training for performing the testing requirements of ASTM A370.
3. A record of annual calibration of testing equipment performed by an independent third party that has standards that are traceable to the National Institute of Standards and Technology and a formal reporting procedure, including published test forms. Calibration records shall be made available for the Engineer's review upon request.

6-02.3(24)I3 Splice Qualification Report

The Contractor shall submit a Splice Qualification Report as a Type 2 Working Drawing. This report shall include, at a minimum:

1. Name of the designated Splicing Quality Control Manager (SQCM).
2. Splice material information
3. Names of the operators who will be performing the splicing
4. Descriptions of the positions, locations, equipment, and procedures that will be used in the splice work.
5. Fabricator's Quality Control Manual for the fabrication of hoops including, but not be limited to, the following:
    a. The pre-production procedures for the qualification of material and equipment.
    b. The methods and frequencies for performing quality control procedures during production.
    c. The calibration procedures and calibration frequency for all equipment.
    d. The welding procedure specification for resistance welding.
    e. The method for identifying and tracking lots.
6. Certifications from the fabricator for qualifications of operators and procedures based on sample qualification tests performed within the past 24 months of the date of the Splice Qualification Report submittal.
    a. Each operator shall be certified by performing two sample splices for each bar size of each splice type that the operator will be performing in the work.
7. Certified test results for all qualification sample splices, tested by an independent qualified testing laboratory and conforming to the specified production test criteria.

6-02.3(24)I4 Production Control Splice Test Criteria

For the purpose of hoop reinforcement splice testing, a lot of splices are defined as 200, or a fraction thereof, of the same type of splice for each bar diameter that is used in the work. A production control sample shall consist of four splices removed from each lot of completed splices.

The Contractor shall select the splices comprising the lot. The Engineer will, or the SQCM shall if the Engineer is not available, select the product control sample of four splices to be tested from each lot.

Production control testing shall be performed for all hoop reinforcement splices used in the work. Production control samples shall be tested in accordance with ASTM A370.

6-02.3(24)I5 Sample Test Criteria

After the splices in a lot have been completed, the SQCM shall notify the Engineer in writing that the splices in this lot conform to the specifications and are ready for testing.

At least one week before sample testing, the Contractor shall notify the Engineer by a Type 1 Working Drawing of the date and location of the testing to allow the Engineer the opportunity to witness the testing.

Samples shall achieve at least 125 percent of the specified yield strength of the bar. In addition, either necking of the bar or a plateau of the stress-strain curve shall be evident at rupture.

6-02.3(24)I6 Sample Acceptance Criteria

If all four sample splices from a lot conform to the requirements of Section 6-02.3(24)I5, all splices in the lot represented by the test will be considered acceptable.

If only two or three of the four sample splices from a lot conform to the requirements of Section 6-02.3(24)I5, the Engineer will, or the SQCM shall if the Engineer is not available, select an additional set of four samples for re-test from the same lot of splices. Should any of the four sample splices from this additional test fail to conform to these requirements; all splices in the lot will be rejected.

Should only one sample splice from a lot conform to the requirements of Section 6-02.3(24)I5, all splices in the lot will be rejected.

Whenever a lot of splices are rejected, the rejected lot and subsequent lots of splices shall not be used in the work until the following requirements are met:

1. The SQCM performs a complete review of the Contractor's quality control process for these splices.
2. A written report is submitted to the Engineer describing the cause of the failure of the splices in this lot and provisions for preventing similar failures in future lots.
3. The Engineer has provided the Contractor with written notification that the report and corrective actions are acceptable.

All bars within a lot shall be visually inspected to verify bar offset at the joint doesn't exceed what is permitted in ANSI/AWS D1.4/D1.4M:2018 Section 6.2.1. All splices with offsets exceeding those as specified in ANSI/AWS D1.4/D1.4M:2018 Section 6.2.1 will be rejected.

6-02.3(24)I7 Reporting Test Results

A Production Control Test Report for all testing performed on each lot shall be prepared by the independent testing laboratory performing the testing and submitted to the SQCM. The report shall include the following information for each test:

1. Contract number.
2. Dates received and tested.
3. Lot number.
4. Bar diameter, hoop diameter, and bar length.
5. Type of splice.
6. Length of test specimen.
7. Physical condition of the test sample splice and description of break and location in relation to splice.
8. Any noticeable defects.
9. Ultimate tensile strength of each splice.

The SQCM shall review, approve with a signature, and submit each Production Control Test Report as a Type 2 Working Drawing. The Contractor shall not encase the splices represented by the report in concrete until receiving the Engineer's written response to the submittal.

6-02.3(24)J Welded Direct Butt Splicing of Hoop Reinforcement for Columns and Shafts

6-02.3(24)J1 Splicing Quality Control Manager

The Contractor shall designate a Splicing Quality Control Manager (SQCM) responsible for the quality control of all hoop reinforcement splicing. The SQCM shall be responsible for preparing all required submittals and reports regarding hoop reinforcement splicing.

Prior to performing any production hoop splicing Work, the Contractor shall submit a Hoop Splicing Quality Control Plan as a Type 2 Working drawing, which shall include the following:

1. Identifying the SQCM. If the SQCM is not a Certified Welding Inspector (CWI), identifying the CWI(s) to be performing all the required inspections
2. Weld Procedure Specification (WPS) and all supporting qualification documents in accordance with ANSI/AWS D1.4/D1.4M:2018
3. Welder qualifications in accordance with ANSI/AWS D1.4/D1.4M:2018.
4. The name of the company and the inspector who will perform the radiographic examinations

Each welder working on production hoop splices shall perform welder qualifications regardless of previous experience or qualification. The WPS and welder qualifications shall be witnessed by a CWI. The Contractor shall notify the Engineer a minimum of 48 hours prior to performing qualification testing to provide the Engineer the opportunity to witness.

6-02.3(24)J2 Welded Direct Butt Splices

Welded direct butt splices shall be complete joint penetration butt welds conforming to ANSI/AWS D1.4/D1.4M:2018 figure 5.2. Split pipe backing shall not be used.

Thermite welding is not allowed.

6-02.3(24)J3 Nondestructive Splice Tests

All splices, including joint geometry and fit-up, in process welding, and completed welds shall be 100 percent visually inspected by the CWI(s) accepted in the hoop Splicing Quality Control Plan. The SQCM shall submit a Type 1 Working Drawing with results of all visual inspections.

Radiographic examinations shall be performed on 25 percent of all complete joint penetration butt welded splices from a lot defined as 200, or a fraction thereof, of the same type of splice for each bar diameter that is used in the work.

The Contractor shall notify the Engineer in writing a minimum of 48 hours before performing radiographic examinations.

All required radiographic examinations shall be performed by the Contractor in accordance with ANSI/AWS D1.4/D1.4M:2018 and as specified below.

Before radiographic examination, welds shall conform to ANSI/AWS D1.4/D1.4M Section 6.4. Radiographic acceptance shall be in accordance with ANSI/AWS D1.4/D1.4M Table 6.1. Acceptance criteria for bar size #7 shall be the same as for bar size #8.

Should more than 12 percent of the splices which have been radiographically examined in any lot be defective, an additional 25 percent of the splices from the same lot, selected by the Engineer, or by the SQCM if the Engineer is not available, shall be radiographically examined. Should more than 12 percent of the cumulative total of splices tested from the same lot be defective, all remaining splices in the lot shall be radiographically examined.

All defects shall be repaired in accordance with ANSI/AWS D1.4/D1.4M, latest edition.

The Contractor shall notify the Engineer in writing a minimum of 48 hours before performing any radiographic examinations.

The radiographic procedure used shall conform to ANSI/AWS D1.1, ANSI/AWS D1.4/D1.4M:2018 Section 9.9, and the following:

1. Two exposures shall be made for each splice. For each of the two exposures, the radiation source shall be centered on each bar to be radiographed. The first exposure shall be made with the radiation source placed at zero degrees from the top of the

weld and perpendicular to the weld root and identified with a station mark of "0". The second exposure shall be at 90 degrees to the "0" station mark and shall be identified with a station mark of "90". When obstructions prevent a 90 degree placement of the radiation source for the second exposure, and when approved in writing by the Engineer, the source may be rotated, around the centerline of the steel reinforcing bar, a maximum of 25 degrees.

2. If more than one weld is to be radiographed during one exposure, the angle between the root line of each weld and the direction to the radiation source shall not be less than 65 degrees.
3. Radiographs shall be made by either X-ray or gamma ray. Radiographs made by X-ray or gamma rays shall have densities of not less than 2.3 nor more than 3.5 in the area of interest. A tolerance of 0.05 in density is allowed for densitometer variations. Gamma rays shall be from the iridium 192 isotope and the emitting specimen shall not exceed 0.18 inches in the greatest diagonal dimension.
4. The radiographic film shall be placed perpendicular to the radiation source at all times; parallel to the root line of the weld unless source placement determines that the film shall be turned; and as close to the root of the weld as possible.
5. The minimum source to film distance shall be maintained so as to ensure that all radiographs maintain a maximum geometric unsharpness of 0.020 at all times, regardless of the size of the steel reinforcing bars.
6. Penetrameters shall be placed on the source side of the bar and perpendicular to the radiation source at all times. One penetrameter shall be placed in the center of each bar to be radiographed, perpendicular to the weld root, and adjacent to the weld. Penetrameter images shall not appear in the weld area.
7. When radiography of more than one weld is being performed per exposure, each exposure shall have a minimum of one penetrameter per bar, or three penetrameters per exposure. When three penetrameters per exposure are used, one penetrameter shall be placed on each of the two outermost bars of the exposure, and the remaining penetrameter shall be placed on a centrally located bar.
8. An allowable weld buildup of 0.16 inch may be added to the total material thickness when determining the proper penetrameter selection. No image quality indicator equivalency will be accepted. Wire penetrameters or penetrameter blocks shall not be used.
9. Penetrameters shall be sufficiently shimmed using a radiographically identical material. Penetrameter image densities shall be a minimum of 2.0 and a maximum of 3.6.
10. Radiographic film shall be Class 1, regardless of the size of the steel reinforcing bars.
11. Radiographs shall be free of film artifacts and processing defects, including, but not limited to, streaks, scratches, pressure marks or marks made for the purpose of identifying film or welding indications.
12. Each splice shall be identified on each radiograph and the radiograph identification and marking system shall be established between the Contractor and the Engineer before radiographic inspection begins. Film shall be identified by lead numbers only; etching, flashing or writing in identifications of any kind will not be permitted. Each piece of film identification information shall be legible and shall include, as a minimum, the following information:
    a. The Contractor's name.
    b. The name of the nondestructive testing firm.
    c. Contract number.
    d. Date of the test.

e. Initials of the radiographer.

f. Part number.

g. Weld number.

The letter "R" and repair number shall be placed directly after the weld number to designate a radiograph of a repaired weld.

13. Radiographic film shall be developed within a time range of one minute less to one minute more than the film manufacturer's recommended maximum development time. Sight development will not be allowed.
14. Processing chemistry shall be done with a consistent mixture and quality, and processing rinses and tanks shall be clean to ensure proper results. Records of all developing processes and chemical changes to the developing processes shall be kept and furnished to the Engineer upon request. The Engineer may request, at any time, that a sheet of unexposed film be processed in the presence of the Engineer to verify processing chemical and rinse quality.
15. The results of all radiographic interpretations shall be recorded on a signed certification and a copy kept with the film packet.

Technique sheets prepared in accordance with ASME Boiler and Pressure Vessels Code Section V Article 2 Section T-291 shall also contain the developer temperature, developing time, fixing duration and all rinse times.

The Contractor shall maintain the radiographs and the radiographic inspection report(s) in the shop until the Engineer reviews them or requests copies. If the Engineer reviews them in the shop then the film and reports shall be released to the Engineer for permanent record keeping at that time. If copies are requested, the Contractor shall submit a Type 2 Working Drawing consisting of the film and a PDF or two paper copies of the radiographic inspection report. Adequate facilities and equipment shall be provided the Engineer for examining film, if performed in the shop.

If the Engineer has not reviewed the film and reports in the shop or requested copies within ten working days of completion of the lot, the Contractor shall submit a Type 2 Working Drawing consisting of the film and reports.

6-02.3(24)K Welded Lap Splicing of Hoop Reinforcement for Shafts

All production splices shall be 100 percent visually inspected for weld quality, size and length.

6-02.3(25) Prestressed Concrete Girders

Precast concrete girders shall be constructed in accordance with Section 6-02.3(9), except as modified in this section.

The manufacturing facility of prestressed concrete girders shall be certified by the Precast/Prestressed Concrete Institute's Plant Certification Program for the type of prestressed member to be produced and shall be approved by WSDOT as a Certified Prestress Concrete Fabricator prior to the start of production. WSDOT certification will be granted at, and renewed during, the annual prestressed plant review and approval process in accordance with WSDOT *Materials Manual* M 46-01.04 Standard Practice QC 6.

The Contracting Agency intends to perform Quality Assurance Inspection. By its inspection, the Contracting Agency intends only to facilitate the Work and verify the quality of that Work. This inspection shall not relieve the Contractor of the responsibility for identifying and replacing defective material or Work.

The various types of prestressed concrete girders are:

Prestressed Concrete I Girder – Refers to a prestressed concrete girder with a flanged I shaped cross section, requiring a cast-in-place concrete deck to support traffic loads. WSDOT standard girders in this category include Series W42G, W50G, W58G, and W74G.

Prestressed Concrete Wide Flange I Girder – Refers to a prestressed concrete girder with an I shaped cross section with wide top and bottom flanges, requiring a cast-in-place concrete deck to support traffic loads. WSDOT standard girders in this category include Series WF36G, WF42G, WF50G, WF58G, WF66G, WF74G, WF83G, WF95G, and WF100G.

Prestressed Concrete Wide Flange Deck Girder – Refers to a prestressed concrete wide flange I girder with extended top flange widths designed to support traffic loads, and designed to be mechanically connected at the flange edges to adjacent girders at the job site. WSDOT standard girders in this category include Series WF39DG, WF45DG, WF53DG, WF61DG, WF69DG, WF77DG, WF86DG, WF98DG, and WF103DG.

Prestressed Concrete Wide Flange Thin Deck Girder – Refers to a prestressed concrete wide flange I girder with extended top flange widths requiring a cast-in-place concrete deck to support traffic loads. Flange edges extend to flange edges of adjacent girders at the job site. WSDOT standard girders in this category include Series WF36TDG, WF42TDG, WF50TDG, WF58TDG, WF66TDG, WF74TDG, WF83TDG, WF95TDG, and WF100TDG.

Prestressed Concrete Deck Bulb Tee Girder – Refers to a prestressed concrete girder with a top flange designed to support traffic loads, and designed to be mechanically connected at the flange edges to adjacent girders at the job site. WSDOT standard girders in this category include Series W35DG, W41DG, W53DG, and W65DG.

Prestressed Concrete Slab Girder – Refers to a prestressed concrete slab girder, with or without voids. Prestressed concrete ribbed section girders and prestressed concrete double tee girders shall conform to the requirements specified for prestressed concrete slab girders.

Prestressed Concrete Tub Girder – Refers to prestressed concrete tub girders with a U shaped cross section, requiring a cast-in-place concrete deck to support traffic loads. WSDOT standard girders in this category include Series U**G* or Series UF**G*, where U specifies webs without top flanges, UF specifies webs with top flanges, ** specifies the girder height in inches, and * specifies the bottom flange width in feet.

Spliced Prestressed Concrete Girder – Refers to prestressed concrete girders initially fabricated in segments which are longitudinally spliced together with cast-in-place concrete closures and post tensioning. Post tensioning materials and construction shall conform to Section 6-02.3(26), except that ducts for prestressed concrete wide flange I girders may be 24-gage, semi-rigid, galvanized, corrugated, ferrous metal. WSDOT prestressed concrete wide flange I girders in this category include Series WF74PTG, WF83PTG, WF95PTG, and WF100PTG. WSDOT prestressed concrete tub girders in this category include Series U**PTG* and UF**PTG* where U, UF, **, and * are as defined for prestressed concrete tub girders.

6-02.3(25)A Shop Drawings

Shop drawings for prestressed concrete girders shall be submitted as Type 2 Working Drawings. The only deviations to the Plans that will be permitted are those approved by the annual plant approval process and those listed below:

1. Addition of inserts for construction purposes including falsework.

2. Small penetrations no larger than 1-inch diameter for construction purposes including overhang bracket supports, deck formwork hangers and temporary girder bracing. Penetrations in top flanges shall be offset from the edge of the flange the minimum distance shown in the Plans.
3. Small penetrations no larger than 2-inch in diameter for girder shipping tie-downs.
4. Small adjustments in girder length to account for elastic shortening, creep and shrinkage
5. Strand adjustments, as long as the center of gravity of the strands remains at the location shown in the plans and concrete cover is not reduced.
6. Diaphragm web hole vertical adjustments to avoid harped strands.
7. Substitution of welded wire reinforcement for conventional reinforcing steel.

Shop drawings shall show the size and location of all inserts and penetrations. Penetrations for deck formwork and falsework shall match the deck formwork Working Drawings. Field-drilled holes in prestressed concrete girders are not allowed.

Deformed welded wire reinforcement conforming to Sections 9-07.7 and 9-07.8 may be substituted for the mild steel reinforcement shown in the plans. The substitution shall be submitted as a Type 2E Working Drawing. The AASHTO LRFD Bridge Design Specification requirements (latest edition including interims) shall be satisfied, including at a minimum the following Articles:

5.8.2.6 Types of Transverse Reinforcement

5.8.2.8 Design and Detailing Requirements

5.10.3 Spacing of Reinforcement

5.10.6.3 Ties

5.10.7 Transverse Reinforcement for Flexural Members

5.10.8 Shrinkage and Temperature Reinforcement

5.10.10 Pretensioned Anchorage Zones

5.11.2.5 Welded Wire Fabric

5.11.2.6.3 Anchorage of Wire Fabric Reinforcement

5.11.6 Splices of Welded Wire Fabric

Yield strengths in excess of 75.0 ksi shall not be used for welded wire reinforcement.

The spacing of vertical welded wire reinforcement within slabs and girder webs shall not exceed 18 inches or the height of the member minus 3 inches, whichever is less. Longitudinal wires and welds are permitted in girder flanges but shall be excluded from girder webs. For vertical welded wire reinforcement in prestressed concrete slab girders, no welded joints other than those required for anchorage shall be permitted. Epoxy-coated wire and welded wire reinforcement shall conform to Section 9-07.3 with the exception that ASTM A884 Class A Type I shall be used instead of ASTM A775.

Shop drawings for spliced prestressed concrete girders shall also conform to Section 6-02.3(26)A. The Working Drawings for spliced prestressed concrete girders shall include all details related to the post-tensioning operations in the field, including details of hardware required, tendon geometry, blockout details, and details of additional or modified steel reinforcing bars required in cast-in-place closures.

6-02.3(25)B Prestressing

Each stressing system shall have a pressure gauge or load cell that will measure jacking force. The gauge shall display pressure accurately and readably with a dial at least 6 inches in diameter or with a digital display. Each jack and its gauge shall be calibrated as a unit and shall be accompanied by a certified calibration chart. The Contractor shall submit a Type 1 Working Drawing consisting of one copy of this chart. The cylinder extension during calibration shall be in approximately the position it will occupy at final jacking force.

Jacks and gauges shall be recalibrated and recertified:

1. Annually,
2. After any repair or adjustment, and
3. Anytime there are indications that the jack calibration is in error.

The Engineer may use load cells to check jacks, gauges, and calibration charts before and during tensioning.

All load cells shall be calibrated and shall have an indicator that shows prestressing force in the strand. The range of this cell shall be broad enough that the lowest 10 percent of the manufacturer's rated capacity will not be used to measure jacking force.

From manufacture to encasement in concrete, prestressing strand shall be protected against dirt, oil, grease, damage, and all corrosives. Strand shall be stored in a dry, covered area and shall be kept in the manufacturer's original packaging until placement in the forms. If prestressing strand has been damaged or pitted, it will be rejected. Prestressing strand with rust shall be spot-cleaned with a nonmetallic pad to inspect for any sign of pitting or section loss. Once the prestressing steel has been installed, no welds or grounds for welders shall be made on the forms or the steel in the girder, except as specified.

When the Plans require temporary strands, they may be pretensioned or post-tensioned. If they are post-tensioned, they shall be stressed on the same day that the permanent prestress is released and prior to lifting the girder or segment. When the Plans require continuous temporary strands for spliced prestressed concrete girders, the girder shall be spliced and the temporary strands shall be post-tensioned prior to lifting the spliced girder.

The Contractor shall be responsible for properly sizing the anchorage plates to prevent bursting or splitting of the concrete due to post-tensioning. The inside diameter of the debonding sleeves for all temporary strands shall be large enough such that the temporary strands fully retract upon cutting. Temporary strands shall be cut or released in accordance with Section 6-02.3(25)L5.

Post-tensioning of spliced prestressed concrete girders shall conform to Section 6-02.3(26) and the following requirements:

1. Before tensioning, the Contractor shall remove all side forms from the cast-in-place concrete closures. From this point until 48 hours after grouting the tendons, the Contractor shall keep all construction and other live loads off the Superstructure and shall keep the falsework supporting the superstructure in place.
2. The Contractor shall not tension the post-tensioning reinforcement until the concrete in the cast-in-place closures reaches the minimum compressive strength specified in the Plans. This strength shall be measured with concrete cylinders made of the same concrete and cured under the same conditions as the cast-in-place closures.
3. All post-tensioning shall be completed before placing the sidewalks and barriers on the Superstructure.

6-02.3(25)C Casting

Side forms shall be steel except that cast-in-place concrete closure forms for spliced prestressed concrete girders, interior forms of prestressed concrete tub girders, and end bulkhead forms of prestressed concrete girders may be wood. Interior voids for prestressed concrete slab girders with voids shall be formed by either wax soaked cardboard or expanded polystyrene forms. The interior void forms shall be secured in the position as shown in the Working Drawings, and shall remain in place.

All concrete mixes to be used shall be preapproved in the WSDOT plant certification process. The temperature of the concrete when placed shall be between 50°F and 90°F.

Slump shall not exceed 4 inches for normal concrete nor 7 inches with the use of a high range water-reducing admixture, nor 9 inches when both a high range water-reducing admixture is used and the water/cement ratio is less than or equal to 0.35. For self-consolidating concrete (SCC), the slump requirements specified above do not apply, and are instead replaced by the target slump flow and slump flow range specified as part of the SCC mix design.

Air-entrainment is not required in the concrete placed into prestressed concrete girders, cast-in-place concrete closures for spliced prestressed concrete girders, and prestressed concrete partial-depth stay-in-place panels, provided that the water/cement ratio is less than or equal to 0.45.

6-02.3(25)C1 Acceptance Testing of Concrete for Prestressed Concrete Girders

Compressive strength cylinders and concrete acceptance testing shall be performed once per prestressed concrete girder or once per fabrication line of prestressed concrete girders. Concrete shall not be placed until fresh concrete testing indicates concrete is within acceptable limits.

Acceptance testing shall be performed by the Contractor and test results shall be submitted to the Engineer. Unless otherwise noted below, the test methods described in Section 6-02.3(5)D shall be followed. Concrete compressive strength shall be in accordance with Section 6-02.3(25)E.

Concrete that is not self-consolidating concrete will be accepted as follows:

1. Temperature within the allowable temperature band.
2. Slump below the maximum allowed.

Concrete that is self-consolidating concrete will be accepted as follows:

1. Temperature within the allowable temperature band.
2. Slump flow within the target slump flow range
3. VSI less than or equal to 1 in accordance with ASTM C1611, Appendix X1, using Filling Procedure B.
4. J ring passing ability less than or equal to 1.5-inches.
5. Rapid assessment of static segregation resistance of self-consolidating concrete using penetration test in accordance with ASTM C1712 shall be less than or equal to 15 mm.

6-02.3(25)D Curing

During curing, the Contractor shall keep the girder in a saturated curing atmosphere until the girder concrete has reached the required release strength. If the Engineer concurs, the Contractor may shorten curing time by heating the outside of impervious forms. Heat may be radiant, convection, conducted steam, or hot air. With steam, the arrangement shall envelop the entire surface with saturated steam. Hot air curing will not be allowed, unless the Contractor submits Type 2 Working Drawings consisting of the proposed method to envelop and maintain the girder in a saturated atmosphere. Saturated atmosphere means a relative humidity of at least 90 percent. The Contractor shall never allow dry heat to touch the girder surface at any point.

Under heat curing methods, the Contractor shall:

1. Keep all unformed girder surfaces in a saturated atmosphere throughout the curing time;

2. Embed a thermocouple (linked with a thermometer accurate to plus or minus 5°F) 6 to 8 inches from the top or bottom of the girder on its centerline and near its midpoint;
3. Monitor with a recording sensor (accurate to plus or minus 5°F) arranged and calibrated to continuously record, date, and identify concrete temperature throughout the heating cycle;
4. Make this temperature record available for the Engineer to inspect;
5. Heat concrete to no more than 100°F during the first 2 hours after placing the concrete, and then increase no more than 25°F per hour to a maximum of 175°F;
6. Cool concrete, after curing is complete, no more than 25°F per hour, to 100°F; and
7. Keep the temperature of the concrete above 60°F until the girder reaches release strength.

The Contractor may strip side forms from prestressed concrete girders once the concrete has reached a minimum compressive strength of 3,000 psi. All damage from stripping is the Contractor's responsibility.

Curing of cast-in-place concrete closures for spliced prestressed concrete girders shall conform to Section 6-02.3(11).

6-02.3(25)E Contractors Control Strength

Concrete strength shall be measured on test cylinders cast from the same concrete as that in the girder. These cylinders shall be cured under time-temperature relationships and conditions that simulate those of the girder. If the forms are heated by steam or hot air, test cylinders will remain in the coolest zone throughout curing. If forms are heated another way, the Contractor shall provide a record of the curing time-temperature relationship for the cylinders for each girder to the Engineer. When two or more girders are cast in a continuous line and in a continuous pour, a single set of test cylinders may represent all girders provided the Contractor demonstrates uniformity of casting and curing to the satisfaction of the Engineer.

The Contractor shall mold, cure, and test enough of these cylinders to satisfy Specification requirements for measuring concrete strength. The Contractor may use 4- by 8-inch or 6- by 12-inch cylinders.

Test cylinders may be cured in a moist room or water tank in accordance with FOP for AASHTO R 100 after the girder concrete has obtained the required release strength. If, however, the Contractor intends to ship the girder prior to the standard 28-day strength test, the design strength for shipping shall be determined from cylinders placed with the girder and cured under the same conditions as the girder. These cylinders may be placed in an unninsulated, moisture-proof envelope.

To measure concrete strength in the girder, the Contractor shall randomly select two test cylinders. The average compressive strength of the two cylinders shall be equal or greater than the specified strength and neither cylinder shall have a compressive strength that is more than 5 percent below the specified strength.

If too few cylinders were molded to carry out all required tests on the girder, the Contractor shall sample and test cores from the girder under the surveillance of the Engineer. Cores sampled from one girder may not be used to represent any other girder, regardless of whether the girders were cast in a continuous line or pour. Cores shall avoid all prestressing strands, steel reinforcing bars and interior voids.

For prestressed concrete slab girders, a test shall consist of four cores measuring 3 inches in diameter by 6 inches in length (for slabs) or by the thickness of the web (for ribbed and double tee sections). Two cores shall be taken from each side of the girder with one on each side of the girder span midpoint, at locations accepted by the Engineer. The core locations for prestressed concrete ribbed and double tee sections shall be immediately beneath the top flange.

For prestressed concrete tub girders, a test shall consist of four cores measuring 3 inches in diameter by the thickness of the web. Two cores shall be taken from each web approximately 3 feet to the left and to the right of the center of the girder span.

For all other prestressed concrete girders, a test shall consist of three cores measuring 3 inches in diameter by the thickness of the web and shall be removed from just below the top flange; one at the midpoint of the girder's length and the other two approximately 3 feet to the left and approximately 3 feet to the right.

The cores shall be taken in accordance with AASHTO T 24 and shall be tested in accordance with AASHTO T 22. The Engineer may accept the girder if the average compressive strength of the all test cores from the girder are at least 85 percent of the specified compressive strength with no one core less than 75 percent of specified compressive strength. If there are more than four cored holes in a girder, the prestressing reinforcement shall not be released until the holes are patched and the patch material has attained a minimum compressive strength equal to the required release compressive strength.

All cored holes shall be patched and cured prior to shipment of the girder. The girder shall not be shipped until tests show the patch material has attained a minimum compressive strength of 4,000 psi.

If the annual plant approval includes procedures for patching cored holes, the cored holes shall be patched in accordance with this procedure. Otherwise, the Contractor shall submit a core hole patching procedure as a Type 2 Working Drawing.

6-02.3(25)F Prestress Release

Side and flange forms that restrain deflection shall be removed before release of the prestressing reinforcement.

All strands shall be released in a way that will minimize eccentricity of the prestressing force about the centerline of the girder. This release shall not occur until tests show each girder has reached the minimum compressive strength required by the Plans.

The Contractor may request permission to release the prestressing reinforcement at a minimum concrete compressive strength less than specified in the Plans. This request shall be submitted as a Type 2E Working Drawing analyzing changes in vertical deflection, girder lateral stability and concrete stresses in accordance with Section 6-02.3(25)L2.

6-02.3(25)G Protection of Exposed Reinforcement

When a girder is removed from its casting bed, all prestressing reinforcement strands projecting from the girder shall be cleaned and painted with a minimum dry film thickness of 1 mil of paint conforming to Section 9-08.1(2)B, and all steel reinforcing bars, including welded wire fabric, projecting from the girder shall be protected in accordance with Section 6-02.3(24)B. During handling and shipping, projecting reinforcement shall be protected from bending or breaking. Just before placing concrete around the painted projecting bars or strands, the Contractor shall remove from them all spattered concrete remaining from girder casting, dirt, oil, and other foreign matter.

6-02.3(25)H Finishing

The Contractor shall apply a Class 1 finish, as defined in Section 6-02.3(14), to:

1. The exterior surfaces of the outside girders; and
2. The bottoms, sides, and tops of the lower flanges on all girders, including the top of the bottom slab between the tub girder webs.

All other girder surfaces shall receive a Class 2 finish.

The interface on girders that contact a cast-in-place concrete deck shall have a finish of dense, screeded concrete without a smooth sheen or laitance on the surface. After vibrating and screeding, and just before the concrete reaches initial set, the Contractor shall texture the interface. This texture shall be applied with a steel brooming tool that etches the surface transversely leaving grooves ⅛ to ¼ inch wide, between ⅛ and ¼ inch deep, and spaced ¼ to ½ inch apart.

On prestressed concrete wide flange deck girders, deck bulb tee girders, ribbed section girders and double tee girders, the Contractor shall test the top surface for flatness and make corrections in accordance with Section 6-02.3(10)D3 except that the straightedge need not exceed the width of the girder top flange when checking the transverse direction. The top surface shall be finished in accordance with Section 6-02.3(10)D6.

The Contractor may repair defects in prestressed concrete girders in accordance with Section 6-01.16.

Prestressed concrete girders exceeding 150.0 feet shall have survey marker embedments or indelible markings at each girder end and at midspan for surveying in accordance with Sections 6-02.3(25)J and 6-02.3(25)K.

6-02.3(25)I Fabrication Tolerances

The girders shall be fabricated as shown in the processed shop drawings and shall meet the dimensional tolerances listed below. Construction tolerances of cast-in-place closures for spliced prestressed concrete girders shall conform to the tolerances specified for spliced prestressed concrete girders. Actual acceptance or rejection will depend on how the Engineer believes a defect outside these tolerances will affect the Structure's strength or appearance:

1. Length:	± ¼ inch per 25 feet of beam length, up to a maximum of ± 1½ inches
2. Width:	
Flanges and webs:	+ ⅜ inch, - ¼ inch
Slab girders:	± ¼ inch
3. Girder Depth (overall):	± ¼ inch
4. Flange Depth:	± ¼ inch
5. Strand Position:	
Individual strands:	± ¼ inch
Bundled strands:	± ½ inch
Harped strand group center of gravity at the girder ends:	± 1 inch
6. Longitudinal Location of Harp Points for Harped	
Strands from Design Locations:	± 20 inches
7. Position of an Interior Void, vertically and horizontally:	± ½ inch
8. Bearing Recess (center of recess to girder end):	± ⅝ inch
9. Girder Ends (deviation from square or designated skew):	
Horizontal:	± ⅛ inch per foot of girder width, up to a maximum of ± ½ inch
Vertical:	± $\frac{3}{16}$ inch per foot of girder depth, up to a maximum of ± 1 inch

10. Bearing Area Deviation from Plane (in length or width of bearing): ± ⅛ inch.
11. Stirrup Reinforcing Spacing: ± 1 inch.
12. Stirrup Projection from Top of Girder:

    Wide flange thin deck and slab girders: ± ½ inch
    All other girders: ± ¾ inch
13. Mild Steel Concrete Cover: - ⅛ inch, + ⅜ inch.
14. Local smoothness of surfaces: ± ¼ inch. in 10 feet
15. Differential Camber between Girders in a Span (measured in place at the job site):

    For wide flange deck and deck bulb tee girders with a cast-in-place reinforced concrete deck:

    Cambers shall be equalized when the differences in cambers between adjacent girders exceeds ± ¾ inch

    For wide flange deck, deck bulb tee and slab girders without a cast-in-place reinforced deck:

    Cambers shall be equalized when the differences in cambers between adjacent girders exceeds ± ¼ inch
16. Position of Inserts for Structural Connections: ± 1 inch.
17. Position of Lifting Embedments: ± 3 inches longitudinal, ± ¼ inch transverse.
18. Weld Ties: ± ½ inch longitudinal, ± ⅛ inch vertical.
19. Position of post tensioning ducts in spliced prestressed concrete girders: ± ¼ inch.
20. Deviation from a smooth curve for post-tensioning ducts at closures based on the sum total of duct placement and alignment tolerances: ± ⅜ inch.

6-02.3(25)J Horizontal Alignment

The Contractor shall check and record the horizontal alignment (sweep) of each girder at the following times:

1. Check Item 1 – Initial, upon removal of the girder from the casting bed;
2. Check Item 2 – Shipment, within 30 days prior to shipment; and
3. Check Item 3 – Erection, after girder erection and cutting temporary top strands but prior to equalization, welding ties or placement of diaphragms.

For each check item, horizontal alignment of the top and bottom flanges shall be checked and recorded. Alternatively, the Contractor may check and record the horizontal alignment of the web near mid-height of the girder. Each check shall be made by measuring the maximum offset at mid-span relative to a chord that starts and stops at the girder ends. The Contractor shall check and record the alignment at a time when the girder is not influenced by temporary differences in surface temperature. For each check item, the alignment shall not be offset more than ⅛ inch for each 10 feet of girder length.

Horizontal alignment measurements for check items 2 and 3 for girders exceeding 150.0 feet in girder plan length shall be accomplished by or under the direct supervision of a licensed surveyor. Check item 2 and 3 reports and submittals for girders exceeding

150.0 feet shall bear the seal of the responsible licensed surveyor certifying the survey measurements. The cost of utilizing a licensed surveyor shall be included in the bid item "Prestressed Concrete Girder" per linear foot. Girders less than 150.0 feet in length do not require check item 2 or 3 measurement by a licensed surveyor.

Records for check item 1 shall be submitted as a Type 1 Working Drawing.

Records for check items 2 and 3 shall be submitted as separate Type 2E Working Drawings when girders exceed 150.0 feet.

Records for check items 2 and 3 shall be submitted as separate Type 2 Working Drawings when girders are 150.0 feet or less.

Records for check item 2, regardless of length, shall be included in the Contractor's prestressed concrete certificate of compliance.

The Engineer will require up to 10 calendar days from the date the check item 2 Working Drawing is received until it is returned to the Contractor. Records for check item 3 will be returned from the Engineer to the Contractor within 5 working days from the date the Working Drawing is received. The Contractor shall not proceed with the Work represented by the Type 2 or 2E Working Drawings until comments from the Engineer have been addressed.

Girders not meeting the tolerance for check item 2 above shall require a plan for horizontal alignment correction, and an analysis of girder lateral stability and stresses in accordance with Section 6-02.3(25)L2. The Contractor shall perform the lateral stability and stress analysis and create the horizontal alignment correction plan and submit them as Type 3E Working Drawings prior to shipment of the girder. The Contractor shall not proceed with the Work represented by the 3E Working Drawing until comments from the Engineer have been addressed

For check item 3, girders that exceed an offset of ⅛ inch for each 10 feet of girder length shall be corrected at the job site to the ⅛ inch maximum offset per 10 feet of girder length before concrete is placed into the diaphragms. The Contractor shall submit a Type 3E Working Drawing for the required corrective action.

Payment for Material on Hand may be made after the check item 1 shows that the girder is within specification as stated above.

The maximum distance between the side of a prestressed concrete slab girder, or the edge of the top flange of a wide flange deck, wide flange thin deck or deck bulb tee girder, and a chord that extends the full length of the girder shall be ± ½ inch after erection (item 3 above).

For girders taller than 3 feet, the Contractor shall check and record the plumbness at the ends of each girder after temporary bracing for erection is in place and prior to erecting subsequent girders.

6-02.3(25)K Vertical Deflection

The Contractor shall check and record the vertical deflection (camber) of each girder at the following times:

1. Check Item 1 – Initial, upon removal of the girder from the casting bed;
2. Check Item 2 – Shipment, within 30 days prior to shipment; and
3. Check Item 3 – Erection, after girder erection and cutting temporary top strands but prior to equalization, welding ties or placement of diaphragms.

For each check item, vertical deflection shall be checked and recorded at each girder end and at midspan. Each check shall be made by measuring the maximum offset at mid-span relative to a chord that starts and stops at the girder ends. The Contractor shall check and record the vertical deflection at a time when the girder is not influenced by temporary differences in surface temperature.

The “D” dimensions shown in the Plans are computed upper and lower bounds of girder vertical deflections at midspan based on a time lapse of 40 and 120 days after release of the prestressing strands. Temporary top strands are assumed to be cut 30 days prior to these elapsed times (10 and 90 days after release of the prestressing strands). The “D” dimensions are intended to advise the Contractor of the expected range of girder vertical deflection at the time of deck placement. A positive (+) “D” dimension indicates upward deflection.

Vertical deflection measurements for check items 2 and 3 for girders exceeding 150.0 feet in girder plan length shall be accomplished by or under the direct supervision of a licensed surveyor. Check item 2 and 3 reports and submittals for girders exceeding 150.0 feet shall bear the seal of the responsible licensed surveyor certifying the survey measurements. The cost of utilizing a licensed surveyor shall be included in the bid item “Prestressed Concrete Girder” per linear foot. Girders less than 150.0 feet in length do not require check item 2 or 3 measurement by a licensed surveyor.

Records for check item 1 shall be submitted as a Type 1 Working Drawing.

Records for check items 2 and 3 shall be submitted as separate Type 2E Working Drawings when girders exceed 150.0 feet.

Records for check items 2 and 3 shall be submitted as separate Type 2 Working Drawings when girders are 150.0 feet or less.

Records for check item 2, regardless of length, shall be included in the Contractor’s prestressed concrete certificate of compliance.

The Engineer will require up to 10 calendar days from the date the check item 2 Working Drawing is received until it is returned to the Contractor. Records for check item 3 will be returned from the Engineer to the Contractor within 5 working days from the date the Working Drawing is received. The Contractor shall not proceed with the Work represented by the Type 2 or 2E Working Drawings until comments from the Engineer have been addressed.

Girders not meeting the “D” dimensions shown in the Plans for check item 2 shall require a plan for vertical deflection correction, and an analysis of girder lateral stability and stresses in accordance with Section 6-02.3(25)L2. The Contractor shall perform the lateral stability and stress analysis and create the vertical deflection correction plan and submit them as Type 3E Working Drawings prior to shipment of the girder. The Contractor shall not proceed with the Work represented by the 3E Working Drawing until comments from the Engineer have been addressed.

For check item 3, if the girder vertical deflection measured is not between the lower “D” dimension bound shown in the Plans and the upper “D” dimension bound shown in the Plans plus ¾ inches, the girder shall be corrected at the job site to the upper “D” dimension plus ¾ inches before concrete is placed into the diaphragms. The Contractor shall submit a Type 3E Working Drawing for the required corrective action.

6-02.3(25)L Handling and Storage

The Contractor shall be responsible for safely lifting, shipping, and erecting prestressed concrete girders.

During handling and storage, each prestressed concrete girder shall always be kept plumb and upright. It shall be lifted only by the lifting embedments (strand lift loops or high-strength threaded steel bars) at either end.

The Contract documents may provide shipping and handling details for girders including lifting embedment locations (L), shipping support locations (L_1 and L_2), minimum shipping support rotational spring constants (K_θ), minimum shipping support center-to-center wheel spacings (W_{cc}), vertical deflections and number of temporary top strands. These shipping and handling details have been determined in accordance with Section 6-02.3(25)L2 and are suggested only.

The Contractor shall submit a Type 2E Working Drawing analyzing girder lateral stability and concrete stresses during lifting, storage, shipping and erection in accordance with Section 6-02.3(25)L2 in the following cases:

1. If any of the analysis assumptions listed in Section 6-02.3(25)L2 are invalid. Determination of validity shall be made by the Contractor, except that analysis assumptions shall be considered invalid if the actual values are outside of the provided tolerances.
2. If the Contractor intends to use shipping and handling configurations or details different than those in the Contract documents, or if the Contractor intends to handle the girder with temporary works or appurtenances attached to the girder. If the only deviation from the listed assumptions is to the location of the shipping support points such that stability is improved, the concrete stresses shall be analyzed, but the lateral stability may be waived.
3. If the Contract documents do not provide shipping and handling details.

6-02.3(25)L1 Lifting and Handling Devices

For strand lift loops in girders with a maximum depth of 3 feet, only ½-inch diameter or 0.6-inch diameter strand conforming to Section 9-07.10 shall be used, and a minimum 2-inch diameter straight pin of a shackle shall be used through the loops. Multiple loops shall be held level in the girder during casting in a manner that allows each loop to carry its share of the load during lifting. The minimum distance from the end of the girder to a single set of strand lift loops shall be 1'-9". When multiple sets of strand lift loops are used, the minimum distance from the end of the girder to the centroid of lift loops shall be 3'-0". The loops for all prestressed concrete girders shall project a minimum of 1′-6″ from the top of the girder. Loops shall extend to within 3 inches clear of the bottom of the girder, terminating with a 9-inch long 90-degree hook. Loads on individual loops shall be limited to 12 kips, and the angle between a lifting cable and the top of the girder shall not be less than 60 degrees.

For high-strength threaded steel bars, a minimum of two 1⅜-inch diameter bars conforming to Section 9-07.11 shall be used at each end of the girder. The lifting hardware that connects to the bars shall be designed, detailed, and furnished by the Contractor. The minimum distance from the end of the girder to the centroid of the lifting bars shall be 3 feet. Lifting bars shall extend to within 3 inches of the bottom of the girder and shall be anchored in the bottom flange with steel plates and nuts. The minimum size of embedded plates for lifting bars shall be ½ inch thick by 3 inches square. Lifting forces on the lifting bars shall not exceed 58 kips on an individual bar and shall be within 10 degrees of perpendicular to the top of the girder.

Lifting loops or embedments for girders with a depth less than 3 feet shall be designed and detailed by the Contractor.

6-02.3(25)L2 Girder Lateral Stability and Stress Analysis

Analysis for girder lateral stability and concrete stresses during lifting, storage, shipping and erection shall be in accordance with the PCI *Recommended Practice for Lateral Stability of Precast*, *Prestressed Concrete Bridge Girders*, First Edition, including March 2020 errata, Publication CB-02-16-E and the AASHTO *LRFD Bridge Design Specifications* edition identified in the Contract documents. The following design criteria shall be met:

1. Factor of Safety against cracking shall be at least 1.0
2. Factor of Safety against failure shall be at least 1.5
3. Factor of Safety against rollover shall be at least 1.5
4. Prestressed concrete girder stresses shall be limited to the following values at all stages of construction and in service:

Condition	Stress	Location	Allowable Stress (ksi)
Temporary Stress at Transfer and Lifting from Casting Bed	Tensile	In areas without bonded reinforcement sufficient to resist the tensile force in the concrete	$0.0948\lambda\sqrt{f'_{ci}} \leq 0.2$
		In areas with bonded reinforcement sufficient to resist the tensile force in the concrete	$0.24\lambda\sqrt{f'_{ci}}$
	Compressive	All areas except at Section extremities when lateral bending is explicitly considered.	$0.65f'_{ci}$
		At section extremities (i.e., flange tips) during handling when lateral bending is explicitly considered	$0.70f'_{ci}$
Temporary Stress at Transfer and Lifting from Casting Bed	Tensile	In areas without bonded reinforcement sufficient to resist the tensile force in the concrete	$0.0948\lambda\sqrt{f'_c}(ksi)$
		In areas with bonded reinforcement sufficient to resist the tensile force in the concrete	$0.24\lambda\sqrt{f'_c}$
	Compressive	All areas except at section extremities when lateral bending is explicitly considered	$0.65f'_c$
		At section extremities (i.e., flange tips) during handling when lateral bending is explicitly considered	$0.70f'_c$
Final Stresses at Service Load	Tensile	All locations	0.0
	Compressive	All areas due to effective prestress and permanent loads	$0.45f'_c$
		All areas due to effective prestress, permanent loads and transient (live) loads	$0.60f'_c$
Final Stresses at Fatigue Load	Compressive	All areas due to the Fatigue I Load Combination plus one-half the sum of effective prestress and permanent loads in accordance with AASHTO LRFD Section 5.5.3.1	$0.40f'_c$

Variables are as defined in the AASHTO LRFD Bridge Design Specifications.

The analysis shall address all effects on girder vertical deflection (camber), "A" dimensions at centerline of bearings and deck screed cambers (C).

Suggested shipping and handling details provided in the Contract documents have been determined using the following analysis assumptions, unless shown otherwise:

1. Girder dimensions, strand locations and lifting embedment locations are within the tolerances specified in Section 6-02.3(25)I.
2. Girder horizontal alignment (sweep) is within the tolerance specified in Section 6-02.3(25)J.
3. Girder vertical deflection (camber) at midspan is less than or equal to the maximum midspan vertical deflection at shipping shown in the Plans.
4. Minimum concrete compressive strength at release (f'ci) has been reached before initial lifting from casting bed. Minimum concrete compressive strength at 28 days (f'c) has been reached before shipping.
5. Lifting lines are plumb.
6. Height of girder bottom above roadway at shipping supports is less than or equal to 72 inches.

7. Height of shipping support roll center above roadway is 24 inches, ± 2 inches.
8. Shipping support longitudinal placement (L_1 and L_2) tolerance is ± 6 inches.
9. Shipping support lateral placement tolerance is ±1 inches.
10. Shipping supports provide the minimum shipping support rotational spring constant (K_θ) and minimum shipping support center-to-center wheel spacings (W_{cc}) shown in the Plans.
11. For shipping at highway speeds a ±20 percent dynamic load allowance (impact) is included with a typical roadway superelevation of 2 percent.
12. For turning at slow speeds, no dynamic load allowance (impact) is included with a maximum roadway superelevation of 6 percent.
13. Wind, centrifugal and seismic forces are not considered.
14. Configurations where a girder is hanging from a lifting line at one end and seated without a lifting line connected at the other end are not considered.
15. Temporary works or appurtenances are not attached to the girder.

6-02.3(25)L3 Girder Storage

If girders are to be stored, the Contractor shall place them on a stable foundation that will keep them in a vertical position. Stored girders shall be supported at the bearing recesses or, if there are no recesses, approximately 2 to 3 feet from the girder ends. After post-tensioning, spliced prestressed concrete girders shall be supported at points between 2 and 5 feet from the girder ends, unless otherwise shown in the Plans. For long-term storage of girders with initial horizontal curvature, the Contractor may wedge one side of the bottom flange, tilting the girders to control curvature. If the Contractor elects to set girders out of plumb during storage, the Contractor shall have the proposed method analyzed by the Contractor's engineer to ensure against damaging the girder.

6-02.3(25)L4 Girder Shipping

After the girder has reached its 28-day design strength, the girder and a completed Certification of Compliance, signed by a Precast/Prestressed Concrete Institute Certified Technician or a Professional Engineer, shall be submitted to the Engineer for inspection. The Certificate of Compliance shall include any alignment correction plans and stability and stress analyses required in Sections 6-02.3(25)J and 6-02.3(25) K, for check items 1 and 2. Girders requiring an alignment correction plan and a stability and stress analysis shall be stamped "Approved For Shipment" by the Engineer following review and approval of the Working Drawing as required in Sections 6-02.3(25)J and 6-02.3(25)K.

No prestressed concrete slab girder shall be shipped for at least 3 days after concrete placement. No prestressed concrete wide flange deck, deck bulb tee or tub girder shall be shipped for at least 7 days after concrete placement, except that they may be shipped 3 days after concrete placement when L/(bd) is less than or equal to 5.0, where L equals the shipping length of the girder, b equals the girder top flange width (for prestressed concrete wide flange deck and deck bulb tee girders) or the bottom flange width (for prestressed concrete tub girders), and d equals the girder depth, all in feet. No other girder shall be shipped for at least 10 days after concrete placement.

Girder support locations during shipping shall be no closer than the girder depth to the ends of the girder at the girder centerline.

If the Contractor elects to assemble spliced prestressed concrete girders into shipping configurations not shown in the Contract documents, the Contractor shall submit a Type 2E Working Drawing analyzing girder lateral stability and concrete stresses in accordance with Section 6-02.3(25)L2 before shipping.

6-02.3(25)L5 Girder Erection

Before erecting prestressed concrete girders, the Contractor shall submit an erection plan as a Type 2E Working Drawing. The erection plan shall provide complete details of the erection process including at a minimum:

1. Temporary falsework support, bracing, guys, gravity anchors, and attachments to other Structure components or objects;
2. Procedure and sequence of operation;
3. Girder stresses during progressive stages of erection in accordance with Section 6-02.3(25)L2;
4. Girder weights, lift points, lifting embedments and devices, spreaders, and angle of lifting cables in accordance with Section 6-02.3(25)L, etc.;
5. Crane(s) make and model, mass, geometry, lift capacity, outrigger size, and reactions;
6. Girder launcher or trolley details and capacity (if intended for use);
7. Locations of cranes, barges, trucks delivering girders, and the location of cranes and outriggers relative to other Structures, including retaining walls and wing walls; and
8. Plans and calculations for all temporary support and bracing systems to resist all anticipated construction loads through construction of the bridge deck.
9. Methods for measuring and adjusting plumbness at girder ends.

The erection plan shall include drawings, notes, catalog cuts, and calculations clearly showing the above listed details, assumptions, and dimensions. Material properties and specifications, structural analysis, and all other data used shall also be included.

The concrete in piers and crossbeams shall reach at least 80 percent of design strength before girders are placed on them.

The Contractor shall hoist girders only by the lifting embedments at the ends, always keeping the girders plumb and upright. When the girders are to receive a cast-in-place concrete deck, lifting embedments shall be removed after erection to provide a minimum 2½-inch clearance to the top of the deck. When the girders are not to receive a cast-in-place concrete deck, lifting embedments shall be removed 1-inch below the girder surface and grouted with an epoxy grout conforming to Section 9-26.3(1)A.

The girders shall be braced in accordance with Sections 6-02.3(17)F4 and 6-02.3(17)F5. When temporary strands in the top flange are used, they shall be cut after the girders are braced and before the girder deflections are equalized and the intermediate diaphragms are cast.

Instead of the oak block wedges shown in the Plans, the Contractor may use Douglas fir blocks if the grain is vertical. The height of oak block wedges at the girder centerline shall not exceed the width.

The Contractor shall fill all block-out holes with a mortar or grout acceptable to the Engineer.

Stop plates and dowel bars for prestressed concrete girders shall be set with either epoxy grout conforming to Section 9-26.3 or type IV epoxy bonding agent conforming to Section 9-26.1.

6-02.3(25)M Girder to Girder Connections

When differential camber between adjacent girders in a span exceeds the tolerance in Section 6-02.3(25)I, the Contractor shall submit a method of equalizing deflections as a Type 1 Working Drawing. All temporary strands in the top flange shall be cut in accordance with Section 6-02.3(25)L5 prior to equalizing girder deflections.

Prestressed concrete girders shall be constructed in the following sequence:

1. If required, deflections shall be equalized in accordance with the Contractor's equalization plan.
2. Intermediate diaphragms shall be placed and all weld ties shall be welded in accordance with Section 6-03.3(25). Welding ground shall be attached directly to the steel plates being welded when welding the weld-ties.
3. All keyways between adjacent girders shown in the Plans to receive grout shall be filled flush with the surrounding surfaces using a grout conforming to Section 9-20.3(2).
4. Equalization equipment shall not be removed and other construction equipment shall not be placed on the structure until intermediate diaphragms and keyway grout have attained a minimum compressive strength of 2,500 psi.

6-02.3(26) Post-Tensioned Concrete

Multistrand post-tensioned structures and grouted post-tensioned structures shall conform to the requirements of *PTI/ASBI M50.3-19 Specification for Multistrand and Grouted Post-Tensioning*. Grouting for post-tensioning systems shall conform to the requirements of *PTI M55.1-19 Specification for Grouting of Post-Tensioned Structures*. Permanent unbonded single-strand post-tensioning shall conform to the requirements of *PTI M10.2-17 Specification for Unbonded Single Strand Tendons*. These specifications shall be collectively referred to as the "PTI requirements." Within the PTI requirements, the term "Construction Engineer" shall be taken as the Engineer.

6-02.3(26)A Post-Tensioning Materials

PTI requirements for materials and components shall not supersede the requirements of Division 9.

Prestressing material shall conform to Section 9-07.10 or Section 9-07.11. Special prestressing materials shall not be used unless specified in the Contract documents.

Post-tensioning systems for multistrand and grouted post-tensioning for permanent structures shall conform to the PTI requirements for Protection Level 2 (PL-2) unless noted otherwise.

Grout for post-tensioning tendons shall be a Grout Type 1 in accordance with Section 9-20.3(1). Non-shrink grout for pourbacks and repairs shall be a Grout Type 2 in accordance with Section 9-20.3(2). Epoxy grout shall conform to the requirements of Section 9-26.3.

Elastomeric coatings shall conform to the requirements of ASTM C836. High molecular weight methacrylate shall confirm to the requirements of Section 6-21.2(3).

The post-tensioning system supplier shall provide a Certificate of Compliance for all components unless otherwise directed by the Engineer.

6-02.3(26)B Personnel

The Contractor shall designate a Direct Supervisor of Post-Tensioning Operations with responsibility over all post-tensioning system installation work. The Contractor shall designate installation & stressing and grouting crew supervisors. Post-Tensioning Operations personnel qualifications shall meet the minimum requirements stated in the PTI requirements, except that personnel qualifications are not required for installation work other than stressing and grouting performed in facilities approved by WSDOT as a Certified Prestress Concrete Fabricator prior to the start of construction.

The post-tensioning system supplier shall designate a Quality Assurance (QA) Manager.

6-02.3(26)C Submittals

Complete post-tensioning installation drawings shall be submitted as Type 2E Working Drawings. Installation drawings shall include system drawings, tendon drawings, anchorage coating system, stressing calculations, proof of PTI-70 certification of the post-tensioning system, post-tensioning anchorage test reports, and proof of qualification for the Direct Supervisor of Post-Tensioning Operations and the Quality Assurance Manager. Post-tensioning anchorage test reports shall include all information necessary for post-tensioning system approval in accordance with the PTI requirements, including the following information for special anchorage device acceptance testing:

1. Dimensions of the test specimen.
2. Working Drawings with details and dimensions of the special anchorage device, including all confining reinforcing steel.
3. Amount and arrangement of supplementary skin reinforcement.
4. Type and yield strength of reinforcing steel.
5. Type and compressive strength of the concrete at the time of testing.
6. Type of testing procedure and all measurements specified for each specimen under the test.

Certifications for all supervisory and crew personnel used to meet the PTI requirements shall be submitted as Type 1 Working Drawings at least 7 days prior to delivery of any post-tensioning system components to the job site, except the qualifications for grouting personnel may be submitted with the written grouting procedures instead.

Written grouting procedure including all items listed in the PTI requirements shall be submitted as a Type 2 Working Drawing.

Jack and gauge unit calibration charts shall be submitted to the Engineer as a Type 1 Working Drawing at least 7 days prior to stressing. If requested by the Engineer, the Contractor shall submit certified calibration reports for load cells used to calibrate each jack and gage combination as a Type 1 Working Drawing.

Stressing records for each tendon shall be submitted to the Engineer as a Type 1 Working Drawing within 3 days after tendon stressing, and prior to grouting.

Grouting records shall be submitted as Type 1 Working Drawings within 3 days after each grouting operation.

Traceability documents and records shall be submitted as a Type 1 Working Drawing upon completion of the post-tensioning system installation.

6-02.3(26)D Design

Tendon locations shown in the Plans indicate final positions after stressing (unless the Plans indicate otherwise).

These stress limits apply to all tendons (unless the Plans set other limits):

1. During jacking prior to seating: less than 90 percent of the yield strength of the steel.
2. At couplers and anchorages immediately after seating: less than 70 percent of the specified tensile strength of the steel.
3. At all other locations along the tendons away from couplers and anchorages immediately after seating: 74 percent of the specified tensile strength of the steel for low relaxation strands, and 70 percent of the specified tensile strength of the steel for all other post-tensioning steel.
4. At service limit state after losses: 80 percent of the yield strength of the steel.

Friction losses used to calculate forces of the post-tensioning steel shall be based on the assumed values used for the design. The assumed anchor set, friction coefficient “μ”, and wobble coefficient “k” values for design are shown in the Plans. The post-tensioning supplier may revise the anchor set value provided all stress limits are satisfied throughout the structure. Tendons less than 20 feet require detailed friction calculations that considers the wedge seating method used.

Anchor devices and local zones shall either be designed in accordance with the AASHTO LRFD Bridge Design Specifications, current edition, or tested as a special anchorage device in accordance with the AASHTO LRFD Bridge Construction Specifications, current edition. Dead ended anchorages will not be permitted unless shown otherwise in the plans. Dead ended anchorages are defined as anchorages that cannot be accessed during the stressing operations.

The structure shall be reinforced with steel reinforcing bars specifically designed and detailed in the anchorage zone in the vicinity of the anchorage device. This reinforcement shall be categorized into two zones. The first or local zone shall be the concrete surrounding and immediately ahead of the anchorage device. The second or general zone shall be the overall anchorage zone, including the local zone.

The steel reinforcing bars required for concrete confinement in the local zone shall be determined by the post-tensioning system supplier and shall be shown in the installation drawings. The calculations shall be included. The local zone steel reinforcing bars shall be furnished and installed by the Contractor, at no additional cost to the Contracting Agency. The steel reinforcing bars required in the general zone shall be as shown in the Plans and are included in the appropriate Bid items.

The Contractor shall determine all points of interference between the mild steel reinforcement and the paths of the post-tensioning tendons. Details to resolve interferences shall be submitted with the installation drawings for acceptance.

6-02.3(26)E General Construction Requirements

The Contractor shall construct supporting falsework in a way that leaves the Superstructure free to contract and lift off the falsework during post-tensioning unless shown otherwise in the Plans. Forms that will remain inside box girders to support the bridge deck shall, by design, resist girder contraction as little as possible.

Before tensioning, the Contractor shall remove all side forms from girders. From this point until 48 hours after grouting the tendons, the Contractor shall keep all construction and other live loads off the Superstructure and shall keep the falsework supporting the Superstructure in place.

All post-tensioning shall be completed as specified in the Contract Plans.

6-02.3(26)F Duct and Component Installation

Vents shall be oriented entirely upwards from the tendon, and drains shall be oriented entirely downward from the tendon.

6-02.3(26)G Prestressing Steel Installation & Stressing

Prestressing strand shall not be spliced. Prestressing bar shall only be spliced at locations shown in the Plans.

Once the prestressing steel is installed, no welds or welding grounds shall be attached to metal forms, structural steel, or reinforcing bars of the structural member.

The Contractor shall not stress the strands until all concrete has reached a compressive strength of at least 4,000 psi unless otherwise shown in the Plans. This strength shall be measured on concrete test cylinders made of the same concrete cured under the same conditions as the cast-in-place unit.

If the prestressing steel will not be stressed and grouted within 7 calendar days after it is placed in the ducts, the Contractor shall apply a corrosion inhibitor.

6-02.3(26)H Grouting

Duct air tests, in accordance with the PTI requirements, shall be performed prior to grouting.

Grout wet density tests shall be performed at the mixer on the first batch of grout and every 2 hours subsequently. Wet density shall be tested at the last outlet of each tendon. The wet density test shall conform to ANSI/API Mud Balance Test, and values shall fall within the range specified by the grout manufacturer.

Grout fluidity tests shall be performed at the mixer on the first batch of grout and every 2 hours subsequently. The fluidity test shall conform to ASTM C939 as modified in the PTI requirements, and the efflux time shall be within 5 seconds of the values established during laboratory testing.

A grout bleed test shall be prior to production grouting for each truckload of prepackaged grout delivered to the job site. The bleed test shall be conducted in accordance with ASTM C1741, with the acceptance criteria of the PTI requirements.

A grout volume change test shall be performed every day of grouting if an expansive admixture is used for external tendons. The volume change test shall conform to ASTM C1090. The vertical height change shall be between 0% and +0.50% at 24 hours and at 28 days.

Grout compressive strength specimens shall be collected every day of grouting. The Contractor shall make grout cubes in accordance with WSDOT T 813 and store the grout cubes in accordance with FOP for AASHTO R 100. Compressive strength testing shall conform to AASHTO T 106. The compressive strength shall be tested at 7 days and 28 days for reference only.

6-02.3(26)I Protection of Anchorages

When anchorages are poured back with pre-packaged materials such as non-shrink grout or epoxy grout, materials shall be placed in accordance with the respective manufacturer's recommendations. Anchorage Coating Systems shall be installed in accordance with the post tensioning installation Working Drawing.

6-02.4 Measurement

Except as noted below, all classes of concrete shall be measured in place by the cubic yard to the neat lines of the Structure as shown in the Plans.

Exception: concrete in cofferdam seals. Payment for Class 4000W concrete used in these seals will be based on the volume calculated using the neatline dimensions for the seal as shown in the Contract Plans. For calculated purposes, the horizontal dimension will be increased by 1 foot outside the seal neatline perimeter. The vertical dimension is the distance between the top and bottom neatline elevations. No payment will be made for concrete that lies outside of these limits to accommodate the Contractor's cofferdam configuration. If the Engineer eliminates the seal in its entirety a Contract change order will be issued.

Exception: concrete in a separate lump-sum, Superstructure Bid item. Concrete quantities noted under this item in the Special Provisions will not be measured. Although the Special Provisions list approximate quantities for the Contractor's convenience, the Contracting Agency does not guarantee the accuracy of these estimates. Before submitting a Bid, the Contractor shall have verified the quantities. Even though actual quantities used may vary from those listed in the Special Provisions, the Contracting Agency will not adjust the lump sum Contract price for Superstructure (except for processed changes).

The Contracting Agency will not measure concrete placed below the established elevation of the bottom of footings or seals.

Lean concrete Type 1 and lean concrete Type 2 will be measured by the cubic yard for the quantity of material placed, except that lean concrete Type 1 or lean concrete Type 2 included in other Contract items will not be measured.

No deduction will be made for pile heads, reinforcing steel, structural steel, bolts, weep holes, rustications, chamfers, edgers, joint filler, junction boxes, miscellaneous hardware, ducts or less than 6-inch diameter drain pipes when computing concrete quantities for payment.

All reinforcing steel will be measured by the computed weight of all steel required by the Plans. The weight of mechanical splices will be based on the weight specified in the manufacturer's existing catalog cut for the specific item. Splices noted as optional in the plans but installed by the Contractor will be included in the measurement. Epoxy-coated bars will be measured before coating. The Contractor shall furnish (without extra allowance):

1. Bracing, spreaders, form blocks, wire clips, and other fasteners.
2. Extra steel in splices not shown in the Plans or specified in the Plans as optional.
3. Extra shear steel at construction joints not shown in the Plans when the Engineer permits such joints for the Contractor's convenience.

The following table shall be used to compute weight of reinforcing steel:

Steel Reinforcing Bar

Deformed Bar Designation Number	Nominal Diameter inches	Unit Weight Pounds per Foot
3	0.375	0.376
4	0.500	0.668
5	0.625	1.043
6	0.750	1.502
7	0.875	2.044
8	1.000	2.670
9	1.128	3.400
10	1.270	4.303
11	1.410	5.313
14	1.690	7.650
18	2.260	13.600

Gravel backfill will be measured as specified in Section 3-07.4.

Expansion joint system strip seal – superstr. and expansion joint system compression seal – superstr. will be measured by the linear foot along its completed line and slope.

Expansion joint modification will be measured by the linear foot of expansion joint modified along its completed line and slope.

Prestressed concrete girder will be measured by the linear foot of girder specified in the Proposal.

Bridge approach slab will be measured by the square yard.

Permeon treatment will be measured by the square yard of concrete surface area receiving the treatment.

Fabricated bearing assemblies (HLMR bearings, fabric pad bearings, transverse restrainer bearings, and pin bearings) will be measured per each for each bearing assembly furnished and installed.

6-02.5 Payment

Payment will be made for each of the following Bid items that are included in the Proposal:

"Conc. Class ____", per cubic yard.

"Commercial Concrete", per cubic yard.

All concrete, except in Superstructure when this is covered by a separate Bid item, will be paid for at the unit Contract price per cubic yard in place for the various classes of concrete. All costs in connection with concrete curing, producing concrete surface finish, and furnishing and applying sealer to concrete surfaces as specified, shall be included in the unit contract price per cubic yard for "Conc. Class ____". If the concrete is to be paid for other than by class of concrete, then the costs shall be included in the associated item of Work.

"Superstructure ___", lump sum.

All costs in connection with constructing, finishing and removing the bridge deck test slab as specified in Section 6-02.3(10)D1 shall be included in the lump sum Contract price for "Superstructure___" or "Bridge Deck___" for one bridge in each project, as applicable.

All costs in connection with providing holes for vents, for furnishing and installing cell drainage pipes for box girder Structures, and furnishing and placing grout and shims under steel shoes shall be included in the unit Contract prices for the various Bid items involved.

All costs in connection with the construction of weep holes, including the gravel backfill for drains surrounding the weep holes except as provided in Section 3-07.4, shall be included by the Contractor in the unit Contract price per cubic yard for "Conc. Class ____".

All costs for Resin Bonded Anchors shall be included in other Bid Items involved.

"Lean Concrete", per cubic yard.

Lean concrete Type 1 or lean concrete Type 2, except when included in another Bid item, will be paid for at the unit Contract price for "Lean Concrete", per cubic yard.

"St. Reinf. Bar ____", per pound.

"Epoxy-Coated St. Reinf. Bar ____", per pound.

Payment for reinforcing steel shall include the cost of drilling holes in concrete for, and setting, steel reinforcing bar dowels with epoxy bonding agent, and furnishing, fabricating, placing, and splicing the reinforcement. In Structures of reinforced concrete where there are no structural steel Bid items, such minor metal parts as expansion joints, bearing assemblies, and bolts will be paid for at the unit Contract price for "St. Reinf. Bar ____" unless otherwise specified.

"Gravel Backfill for Foundation Class A", per cubic yard.

"Gravel Backfill for Foundation Class B", per cubic yard.

"Gravel Backfill for Wall", per cubic yard.

"Deficient Strength Conc. Price Adjustment", by calculation.

"Deficient Strength Conc. Price Adjustment" shall be calculated and paid for as described in Section 6-02.3(5)L. For the purpose of providing a common Proposal for all Bidders, the Contracting Agency has entered an amount for the item "Deficient Strength Conc. Price Adjustment" in the Bid Proposal to become a part of the total Bid by the Contractor. The item "Deficient Strength Conc. Price Adjustment" covers all applicable classes of concrete.

"Expansion Joint System Compression Seal - Superstr.", per linear foot.

"Expansion Joint System Strip Seal - Superstr.", per linear foot.

"Expansion Joint Modification - ___", per linear foot.

"___Bearing - Superstr.", per each.

The unit Contract price per each for "___Bearing - Superstr." shall be full pay for performing the Work as specified in Section 6-02.3(19), including design, testing, and inspection.

"Modular Expansion Joint System___", lump sum.

The lump sum Contract price for "Modular Expansion Joint System___" shall be full pay for performing the Work as specified in Section 6-02.3(13)C.

"Prestressed Conc. Girder ___", per linear foot.

"Bridge Approach Slab", per square yard.

The unit Contract price per square yard for "Bridge Approach Slab" shall be full pay for providing, placing, and compacting the crushed surfacing base course, furnishing and placing Class 4000A concrete, and furnishing and installing compression seal, anchors, and reinforcing steel.

"Permeon Treatment", per square yard.

The unit contract price per square yard for "Permeon Treatment" shall be full pay for performing the work as specified.

6-03 Steel Structures

6-03.1 Description

This Work consists of furnishing, fabricating, erecting, cleaning, and painting steel Structures and the structural steel parts of nonsteel Structures

6-03.2 Materials

Materials shall meet the requirements of the following sections:

Structural Steel and Related Materials	9-06
Paints and Related Materials	9-08
Grout	9-20.3

Structural steel shall be classified as:

1. Structural carbon steel (to be used whenever the Plans do not specify another classification),
2. Structural low alloy steel, and
3. Structural high-strength steel.

Unless the Plans or Special Provisions state otherwise, the following shall be classified as structural carbon steel: shims; ladders; stairways; anchor bolts and sleeves; pipe, fittings, and fastenings used in handrails; and other metal parts, even if made of other materials, for which payment is not specified.

All AASHTO M270 material used in what the Plans show as main load-carrying tension members or as tension components of flexural members shall meet the Charpy V-notch requirements of AASHTO M270 temperature zone 2. All AASHTO M270 material used in what the Plans show as fracture critical members shall meet the Charpy V-notch requirements of AASHTO M270, Fracture Critical Impact Test Requirements, temperature zone 2. Charpy V-notch requirements for other steel materials shall be as specified in the Plans and Special Provisions.

The Contractor shall submit Type 1 Working Drawings describing the methods for visibly marking the material so that it can be traced. These marks shall remain visible at least through the fit-up of the main load-carrying tension members. The marking method shall permit the Engineer to verify: (1) material Specification designation, (2) heat number, and (3) material test reports to meet any special requirements.

For steel in main load-carrying tension members and in tension components of flexural members, the Contractor shall include the heat numbers on the reproducible copies of the as-built shop plans.

6-03.3 Construction Requirements

Structural steel fabricators of plate and box girders, floorbeams, truss members, stringers, cross frames, diaphragms, and laterals shall be certified under the AISC Certification Program for Steel Bridge Fabricators, Advanced Bridges Category unless otherwise specified in the Special Provisions. When fracture critical members are specified in the Contract, structural steel fabricators shall also meet the supplemental requirements F, Fracture Critical, under the AISC Quality Certification Program for Steel Bridge Fabricators.

6-03.3(1) Vacant

6-03.3(2) Facilities for Inspection

The Contractor shall provide all facilities the Inspector requires to inspect material and Work. Inspectors shall be given safe and free access to all areas in the mill and shop.

6-03.3(3) Inspector's Authority

The Inspector may reject materials or Work that does not comply with these Specifications. In any dispute, the Contractor may appeal to the Engineer whose decision shall be final.

By its inspection at the mill and shop, the Contracting Agency intends only to facilitate the Work and prevent errors. This inspection shall not relieve the Contractor of any responsibility for identifying and replacing defective material or Work.

6-03.3(4) Rejections

Even if the Inspector accepts materials or finished members, the Contracting Agency may later reject them if defective. The Contractor shall promptly replace or make good any rejected materials or Work.

6-03.3(5) Mill Orders and Shipping Statements

The Contractor shall furnish as many copies of mill orders and shipping statements as the Engineer requires.

6-03.3(6) Weighing

Structural steel need not be weighed unless the Plans or Special Provisions require it. When a weight is required, it may either be calculated or obtained by scales. The Contractor shall furnish as many copies of the calculations or weight slips as the Engineer requires. If scale weights are used, the Contractor shall record separately the weights of all tools, erection material, and dunnage.

6-03.3(7) Shop Plans

The Contractor shall submit all shop detail plans for fabricating the steel as Type 2 Working Drawings.

If these plans will be submitted directly from the fabricator, the Contractor shall so notify the Engineer in writing.

No material shall be fabricated until: (1) the Working Drawing review is complete, and (2) the Engineer has accepted the materials source.

Before physical completion of the project, the Contractor shall furnish the Engineer one set of reproducible copies of the as built shop plans. The reproducible copies shall be clear, suitable for microfilming, and on permanent sheets that measure no smaller than 11 by 17 inches. Alternatively, the shop drawings may be provided in an electronic format with the approval of the Engineer.

6-03.3(7)A Erection Methods

Before beginning to erect any steel Structure, the Contractor shall submit Type 2E Working Drawings consisting of the erection plan and procedure describing the methods the Contractor intends to use.

The erection plan and procedure shall provide complete details of the erection process including, at a minimum, the following:

1. Temporary falsework support, bracing, guys, gravity anchors, and attachments to other Structure components or objects;
2. Procedure and sequence of operation;
3. Girder stresses during progressive stages of erection;
4. Girder masses, lift points, and lifting devices, spreaders, glommers, etc.;
5. Crane(s) make and model, mass, geometry, lift capacity, outrigger size and reactions;

6. Girder launcher or trolley details and capacity (if intended for use); and
7. Locations of cranes, barges, trucks delivering girders, and the location of cranes and outriggers relative to other Structures, including retaining walls and wing walls.

As part of the erection plan Working Drawings, the Contractor may submit details of an engineered and fabricated lifting bracket bolted to the girder top flanges providing the following requirements are satisfied:

1. The lifting bracket shall be engineered and supporting calculations shall be submitted with the erection plan;
2. The calculations shall include critical stresses in the girder including local stresses in the flanges at lifting bracket locations;
3. The calculations shall include computation of the lifting bracket and associated bolt hole locations and the expected orientation of the girder during picking operation;
4. The lifting bracket shall be load tested and certified for a load at least 2 times the working load and at all angles it will be used (angle of load or rigging). Certification documentation from a previous project may be submitted;
5. Bolt holes in girders added for the lifting bracket connections shall be shown in the shop plans and shall be drilled in the shop. Field drilling of bolt holes for lifting brackets will not be permitted;
6. Bolt holes in girder top flanges shall be filled with high strength bolts after erection in accordance with Section 6-02.3(17)K.

The erection plan shall include drawings, notes, catalog cuts, and calculations clearly showing the above listed details, assumptions, and dimensions. Material properties, Specifications, structural analysis, and any other data used shall also be included.

6-03.3(8) Substitutions

The Contractor shall not substitute sections that differ from Plan dimensions unless the Engineer approves in writing. If the Contractor requests and receives approval to substitute heavier members, the Contracting Agency shall not pay any added cost.

6-03.3(9) Handling, Storing, and Shipping of Materials

Markings applied at the mill shall distinguish structural low alloy steel from structural carbon steel. The fabricator shall keep the two classes of steel carefully separated.

Before fabrication, all material stored at the fabricating plant shall be protected from rust, dirt, oil, and other foreign matter. The Contracting Agency will accept no rust-pitted material.

After fabrication, all material awaiting shipment shall be subject to the same storage requirements as unfabricated material.

All structural steel shall arrive at the job in good condition. As the Engineer requires, steel damaged by salt water shipment shall be thoroughly cleaned by high pressure water flushing, chemical cleaning, or sandblasting, and repainted with the specified shop coat.

All material shall be stored so as to prevent rust and loss of small parts. Piled material shall not rest on the ground or in water but on skids or platforms.

The loading, transporting, unloading, and piling of the structural steel material shall be so conducted that the metal will be kept clean and free from injury from rough handling.

In field assembly of structural parts, the Contractor shall use methods and equipment not likely to twist, bend, deform, or otherwise injure the metal. Any member slightly bent or twisted shall be corrected before it is placed. The Contracting Agency will reject any member with serious handling damage.

Girder sections shall be handled so as to prevent damage to the girders. If necessary, the Contractor shall provide temporary stiffeners to prevent buckling during erection.

6-03.3(10) Straightening Bent Material

If the Engineer permits in writing, plates, angles, other shapes, and built-up members may be straightened. Straightening methods shall not fracture or injure the metal. Distorted members shall be straightened mechanically. A limited amount of localized heat may be applied only if carefully planned and supervised, and only in accordance with the heat-straightening procedure Working Drawing submittal.

Parts to be heat-straightened shall be nearly free from all stress and external forces except those that result from the mechanical pressure used with the heat.

After straightening, the Contractor shall inspect the member for fractures using a method proposed by the Contractor and accepted by the Contracting Agency.

The Contracting Agency will reject metal showing sharp kinks and bends.

The procedure for heat straightening of universal mill (UM) plates by the mill or the fabricator shall be submitted as a Type 2 Working Drawing.

6-03.3(11) Quality of Construction and Finish

Work and finish shall be first-class, equaling the best practice in modern bridge fabrication shops. Welding, shearing, burning, chipping, and grinding shall be done neatly and accurately. All parts of the Work exposed to view shall be neatly finished.

Wherever the Plans show a surface finish symbol, the surface shall be machined.

6-03.3(12) Falsework

All falsework shall meet the requirements of Section 6-02.

6-03.3(13) Fabricating Tension Members

Plates for main load-carrying tension members or tension components of flexural members shall be:

1. Blast cleaned entirely or blast cleaned on all areas within 2 inches of welds to SSPC-SP6, Commercial Blast Cleaning; and
2. Fabricated from plate stock with the primary rolling direction of the stock parallel to the length of the member, or as shown in the Plans.

6-03.3(14) Edge Finishing

All rolled, sheared, and thermal cut edges shall be true to line and free of rough corners and projections. Corners along exposed sheared or cut edges shall be broken by light grinding or another method acceptable to the Engineer to achieve an approximate 1⁄16-inch chamfer or rounding.

Sheared edges on plates more than ⅝ inch thick shall be planed, milled, ground, or thermal cut to a depth of at least ⅛ inch.

Re-entrant corners or cuts shall be filleted to a minimum radius of 1 inch.

Exposed edges of main load-carrying tension members or tension components of flexural members shall have a surface roughness no greater than 250-micro inches as defined by the American National Standards Institute, ANSI B46.1, Surface Texture. Exposed edges of other members shall have surface roughness no greater than 1,000-micro inches.

The Rockwell hardness of thermal-cut edges of structural low alloy or high-strength steel flanges, as specified in Sections 9-06.2 and 9-06.3, for main load-carrying tension members or tension components of flexural members shall not exceed RHC 30. The fabricator shall prevent excessive hardening of flange edges through preheating or post heating.

Hardness testing shall consist of testing thermal-cut edges with a portable hardness tester. The hardness tester, and its operating test procedures, shall be submitted as a Type 1 Working Drawing. The hardness tester shall be convertible to Rockwell C scale values.

At two locations, two tests shall be performed on each thermal-cut edge, one each within ¼ inch of the top and bottom surfaces. The tests shall be located ¼ the length of each thermal-cut edge from each end of the cut. If one or more readings are greater than RHC 30, the entire length of the edge shall be ground or machined to a depth sufficient to provide acceptable readings upon further retests. If thermal-cutting operations conform to procedures established by the steel manufacturer, and hardness testing results are consistently within acceptable limits, the Engineer may authorize a reduction in the testing frequency.

6-03.3(15) Planing of Bearing Surfaces

Ends of columns that bear on base and cap plates shall be milled to true surfaces and accurate bevels.

When assembled, caps and base plates of columns and the sole plates of girders and trusses shall have a fit tolerance within 1⁄32 inch for 75 percent of the contact area. For sole plates, contact area shall be 75 perfcent of the projected area of the web and bearing stiffeners. If warped or deformed, the plates shall be heat straightened, planed, or corrected in some other way to produce accurate, even contact. If necessary for proper contact, bearing surfaces that will contact other metal surfaces shall be planed or milled. Surfaces of warped or deformed base and sole plates that will contact masonry shall be rough finished.

On the surface of expansion bearings, the cut of the planer shall be in the direction of expansion.

Where finish to bear is specified in the Plans, the bearing end of the stiffener shall be flush and square with the flange and shall have at least 75 percent of this area in contact with the flange.

6-03.3(16) Abutting Joints

Abutting ends of compression members shall be faced accurately so that they bear evenly when in the Structure. On built-up members, the ends shall be faced or milled after fabrication.

Ends of tension members at splices shall be rough finished to produce neat, close joints. A contact fit is not required.

6-03.3(17) End Connection Angles

On floorbeams and stringers, end connection angles shall be flush with each other and set accurately in relationship to the position and length of the member. Unless the Plans require it, end connection angles shall not be finished. If, however, faulty assembly requires them to be milled, milling shall not reduce thickness by more than 1⁄16 inch.

6-03.3(18) Built Members

The various pieces forming one built member shall be straight and close fitting, true to detailed dimensions, and free from twists, bends, open joints, or other defects.

When fabricating curved girders, localized heat or the use of mechanical force shall not be used to bend the girder flanges about an axis parallel to girder webs.

6-03.3(19) Hand Holes

Hand holes, whether punched or cut with burning torches, shall be true to sizes and shapes shown in the Plans. Edges shall be true to line and ground smooth.

6-03.3(20) Lacing Bars

Unless the Plans state otherwise, ends of lacing bars shall be neatly rounded.

6-03.3(21) Plate Girders

6-03.3(21)A Web Plates

If web plates are spliced, gaps between plate ends shall be set at shop assembly to measure ¼ inch, and shall not exceed ⅜ inch.

6-03.3(21)B Vacant

6-03.3(21)C Web Splices and Fillers

Web splice plates and fillers under stiffeners shall fit within ⅛ inch at each end. In lieu of the steel material specified in the Plans or Special Provisions, the Contractor may substitute ASTM A1008 or ASTM A1011 steel for all filler plates less than ¼ inch thickness, provided that the grade of filler plate steel meets or exceeds that of the splice plates.

6-03.3(22) Eyebars

Eyebars shall be straight, true to size, and free from twists or folds in the neck or head and from any other defect that would reduce their strength. Heads shall be formed by upsetting, rolling, or forging. Dies in use by the manufacturer may determine the shape of bar heads if the Engineer approves. Head and neck thickness shall not overrun by more than 1⁄16 inch. Welds shall not be made in the body or head of any bar.

Each eyebar shall be properly annealed and carefully straightened before it is bored. Pinholes shall be located on the centerline of each bar and in the center of its head. Holes in bar ends shall be so precisely located that in a pile of bars for the same truss panel the pins may be inserted completely without driving. All eyebars made for the same locations in trusses shall be interchangeable.

6-03.3(23) Annealing

All eyebars shall be annealed by being heated uniformly to the proper temperature, then cooled slowly and evenly in the furnace. At all stages, the temperature of the bars shall be under full control.

Slight bends on secondary steel members may be made without heat. Crimped web stiffeners need no annealing.

6-03.3(24) Pins and Rollers

Pins and rollers shall be made of the class of forged steel the Plans specify. They shall be turned accurately to detailed dimensions, smooth, straight, and flawless. The final surface shall be produced by a finishing cut.

Pins and rollers 9 inches or less in diameter may either be forged and annealed or made of cold-finished carbon steel shafting.

Pins more than 9 inches in diameter shall have holes at least 2 inches in diameter bored longitudinally through their centers. Pins with inner defects will be rejected.

The Contractor shall provide pilot and driving nuts for each size of pin unless the Plans state otherwise.

6-03.3(24)A Boring Pin Holes

Pin holes shall be bored true to detailed dimensions, smooth and straight, and at right angles to the axis of the member. Holes shall be parallel with each other unless the Plans state otherwise. A finishing cut shall always be made.

The distance between holes shall not vary from detailed dimensions by more than 1⁄32 inch. In tension members, this distance shall be measured from outside to outside of holes; in compression members, inside to inside.

6-03.3(24)B Pin Clearances

Each pin shall be 1⁄50 inch smaller in diameter than its hole. All pins shall be numbered after being fitted into their holes in the assembled member.

6-03.3(25) Welding and Repair Welding

Welding and repair welding of all steel bridges, including pedestrian Structures and steel Ferry Terminal Structures, shall comply with the AASHTO/AWS D1.5M/D1.5, latest edition, Bridge Welding Code. Welding and repair welding for all other steel fabrication shall comply with the AWS D1.1/D1.1M, latest edition, Structural Welding Code. The requirements described in the remainder of this section shall prevail whenever they differ from either of the above welding codes.

The Contractor shall weld structural steel only to the extent shown in the Plans. No welding, including tack and temporary welds shall be done in the shop or field unless the location of the welds is shown on the approved shop drawings reviewed and accepted by the Engineer.

Welding procedures shall accompany the shop drawing Working Drawing submittal. The procedures shall specify the type of equipment to be used, electrode selection, preheat requirements, base materials, and joint details. When the procedures are not prequalified by AWS or AASHTO, evidence of qualification tests shall be submitted. Steel bridge elements not subject to live load tensile stress and not welded to main members in tension areas, such as steel expansion dams, bearings, handrail, drainage components, and curb plates, may follow the requirements of AASHTO/AWS D1.5M/D1.5, latest edition, Bridge Welding Code for Welding of Ancillary Products whereby qualification of welding procedures is not required.

Welding shall not begin until completion of the shop plan Working Drawing review as required in Section 6-03.3(7). These plans shall include procedures for welding, assembly, and any heat-straightening or heat-curving.

Any welded shear connector longer than 8 inches may be made of two shorter shear connectors joined with full-penetration welds.

In shielded metal-arc welding, the Contractor shall use low-hydrogen electrodes.

In submerged-arc welding of main load-carrying tension members and tension components of flexural members, flux shall be oven-dried at 550°F for at least 2 hours, then stored in ovens held at 250°F or more. If not used within 4 hours after removal from a drying or storage oven, flux shall be redried before use.

Preheat and interpass temperatures shall conform to the applicable welding code as specified in this section. When welding main members of steel bridges, the minimum preheat shall not be less than 100°F.

If groove welds (web-to-web or flange-to-flange) have been rejected, they may be repaired no more than twice. If a third failure occurs, the Contractor shall:

1. Trim the members, if the Engineer concurs, at least ½ inch on each side of the weld; or
2. Replace the members at no expense to the Contracting Agency.

By using extension bars and runoff plates, the Contractor shall terminate groove welds in a way that ensures the soundness of each weld to its ends. The bars and plates shall be removed after the weld is finished and cooled. The weld ends shall then be ground smooth and flush with the edges of abutting parts.

The Contractor shall not:

1. Weld with electrogas or electroslag methods unless shown in the Plans or allowed by the Engineer,
2. Weld nor flame cut when the ambient temperature is below 20°F, or
3. Use coped holes in the web for welding butt splices in the flanges unless the Plans show them.

6-03.3(25)A Welding Inspection

The Contractor's inspection procedures, techniques, methods, acceptance criteria, and inspector qualifications for welding of steel bridges, including pedestrian Structures, shall be in accordance with the AASHTO/AWS D1.5M/D1.5, latest edition, Bridge Welding Code. The Contractor's inspection procedures, techniques, methods, acceptance criteria, and inspector qualifications for welding of steel Structures other than steel bridges, including pedestrian Structures, shall be in accordance with AWS D1.1/D1.1M, latest edition, Structural Welding Code. The requirements described in the remainder of this section shall prevail whenever they differ from either of the above welding codes.

Nondestructive testing in addition to visual inspection shall be performed by the Contractor. Unless otherwise shown in the Plans or specified in the Special Provisions, the extent of inspection shall be as specified in this section. Testing and inspection shall apply to welding performed in the shop and in the field.

After the Contractor's welding inspection is complete, the Contractor shall allow the Engineer sufficient time to perform quality assurance welding inspection.

6-03.3(25)A1 Visual Inspection

All welds shall be 100 percent visually inspected. Visual inspection shall be performed before, during, and after the completion of welding.

6-03.3(25)A2 Radiographic Inspection

Complete penetration tension groove welds in Highway bridges, pedestrian Structures, and steel Ferry Terminal Structures shall be 100 percent radiographically inspected. These welds include those in the tension area of webs, where inspection shall cover the greater of these two distances: (a) 15 inches from the tension flange, or (b) ⅓ of the web depth. In addition, edge blocks conforming to the requirements of AASHTO/AWS D1.5M/D1.5, latest edition, Bridge Welding Code Section 8.10.14 shall be used for radiographic inspection.

The Contractor shall maintain the radiographs and the radiographic inspection report in the shop until the last joint to be radiographed in that member is accepted by the radiographer representing the Contractor. Within 2 working days following this acceptance, the Contractor shall submit the film, or electronic version with associated viewing program, and one copy of the radiographic inspection report to the Materials Engineer, Department of Transportation, PO Box 47365, Olympia, WA 98504-7365.

6-03.3(25)A3 Ultrasonic Inspection

Complete penetration groove welds on plates 5⁄16 inch and thicker in the following welded assemblies or Structures shall be 100 percent ultrasonically inspected:

1. Welded connections and splices in Highway bridges, including pedestrian Structures, and earth retaining Structures, excluding longitudinal butt joint welds in beam or girder webs.
2. Bridge bearings and modular expansion joints.
3. Sign bridges, cantilever sign Structures, and bridge mounted sign brackets excluding longitudinal butt joint welds in beams.

4. Light, signal, and strain pole standards, as defined in Section 9-29.6.
5. Washington State Ferries steel terminal Structures.

A minimum of 30 percent of complete penetration vertical welds on steel column jackets ⁵⁄₁₆-inch and thicker, within 1.50 column jacket diameters of the top and bottom of each column, shall be inspected. If any rejectable flaws are found, 100 percent of the weld within the specified limits shall be inspected. The largest column cross section diameter for tapered column jackets shall constitute one column jacket diameter.

The testing procedure and acceptance criteria for tubular members shall conform to the requirements of the AWS D1.1/D1.1M latest edition, Structural Welding Code.

6-03.3(25)A4 Magnetic Particle Inspection

1. Fillet and partial penetration groove welds:

   At least 30 percent of each size and type of fillet welds (excluding intermittent fillet welds) and partial penetration groove welds in the following welded assemblies or Structures shall be tested by the magnetic particle method:

   a. Flange-to-web connections in Highway bridges and pedestrian Structures.
   b. End and intermediate pier diaphragms in Highway bridges and pedestrian Structures.
   c. Stiffeners and connection plates in Highway bridges and pedestrian Structures.
   d. Welded connections and splices in earth retaining Structures.
   e. Boxed members of trusses.
   f. Bridge bearings and modular expansion joints.
   g. Sign bridges, cantilever sign Structures, and bridge mounted sign brackets.
   h. Light, signal, and strain pole standards, as defined in Section 9-29.6.
   i. Washington State Ferries steel terminal Structures.

2. Longitudinal butt joint welds in beam and girder webs:

   At least 30 percent of each longitudinal butt joint weld in the beam and girder webs shall be tested by the magnetic particle method.

3. Complete penetration groove welds on plates less than ⁵⁄₁₆ inch (excluding steel column jackets) shall be 100 percent tested by the magnetic particle method. Testing shall apply to both sides of the weld, if backing plate is not used.
4. The ends of each complete penetration groove weld at plate edges shall be tested by the magnetic particle method.
5. A minimum of 30 percent of complete penetration vertical welds on steel column jackets less than ⁵⁄₁₆ inch, within 1.50 column jacket diameters of the top and bottom of each column, shall be magnetic particle inspected. The largest column cross section diameter for tapered column jackets shall constitute one column jacket diameter.

Where 100 percent testing is not required, the Engineer reserves the right to select the location(s) for testing.

If rejectable flaws are found in any test length of weld in item 1 or 2 above, the full length of the weld or 5 feet on each side of the test length, whichever is less, shall be tested. If any rejectable flaws are found in any test length of item 4 above, 100 percent of the weld within the specified limits shall be inspected.

6-03.3(26) Screw Threads

Screw threads shall be U.S. Standard and shall fit closely in the nuts.

6-03.3(27) High-Strength Bolt Holes

At the Contractor's option under the conditions described in this section, holes may be punched or subpunched and reamed, drilled or subdrilled and reamed, or formed by numerically controlled drilling operations.

The hole for each high-strength bolt shall be $\frac{1}{16}$ inch larger than the nominal diameter of the bolt.

In fabricating any connection, the Contractor may subdrill or subpunch the holes then ream full size after assembly or drill holes full size from the solid with all thicknesses of material shop assembled in the proper position. If the Contractor chooses not to use either of these methods, then the following shall apply:

1. Drill bolt holes in steel splice plates full size using steel templates.
2. Drill bolt holes in the main members of trusses, arches, continuous beam spans, bents, towers, plate girders, box girders, and rigid frames at all connections as follows:
   a. A minimum of 30 percent of the holes in one side of the connection shall be made full size using steel templates.
   b. A minimum of 30 percent of the holes in the second side shall be made full size assembled in the shop.
   c. All remaining holes may be made full size in unassembled members using steel templates.
3. Drill bolt holes in crossframes, gussets, lateral braces, and other secondary members full size using steel templates.

The Contractor shall submit Type 2 Working Drawings consisting of a detailed outline of the procedures proposed to accomplish the Work from initial drilling through shop assembly.

6-03.3(27)A Punched Holes

For punched holes, die diameter shall not exceed punch diameter by more than $\frac{1}{16}$ inch. Any hole requiring enlargement to admit the bolt shall be reamed. All holes shall be cut clean with no torn or ragged edges. The Contracting Agency will reject components having poorly matched holes.

6-03.3(27)B Reamed and Drilled Holes

Reaming and drilling shall be done with short taper reamers or twist drills, producing cylindrical holes perpendicular to the member. Reamers and drills shall be directed mechanically, not hand-held , except hand-held reamers may be used under the following conditions:

1. For preparing existing holes to accept bolts during rivet replacement on existing steel bridge painting and retrofit projects
2. Hand-held reamers shall be power driven with tapered bridge reamers as accepted by the Engineer
3. Unless otherwise shown in the Plans, holes shall be 1/16 inch larger than the bolt diameter specified in the Plans

Connecting parts that require reamed or drilled holes shall be assembled and held securely as the holes are formed, then match-marked before disassembly. The Contractor shall provide the Engineer a diagram showing these match-marks. The Contracting Agency will reject components having poorly matched holes.

Burrs on outside surfaces shall be removed. If the Engineer requires, the Contractor shall disassemble parts to remove burrs.

If templates are used to ream or drill full-size connection holes, the templates shall be positioned and angled with extreme care and bolted firmly in place. Templates for reaming or drilling matching members or the opposite faces of one member shall be duplicates. All splice components shall be match-marked unless otherwise approved by the Engineer.

6-03.3(27)C Numerically Controlled Drilled Connections

In forming any hole described in Section 6-03.3(27), the fabricator may use numerically controlled (N/C) drilling or punching equipment if it meets the requirements in this Subsection.

The Contractor shall submit Type 1 Working Drawings consisting of a detailed outline of proposed N/C procedures. This outline shall:

1. Cover all steps from initial drilling or punching through check assembly;
2. Include the specific members of the Structure to be drilled or punched, hole sizes, locations of the common index and other reference points, makeup of check assemblies, and all other information needed to describe the process fully.

N/C holes may be drilled or punched to size through individual pieces, or may be drilled through any combination of tightly clamped pieces.

When the Engineer requires, the Contractor shall demonstrate that the N/C procedure consistently produces holes and connections meeting the requirements of these Specifications.

6-03.3(27)D Accuracy of Punched, Subpunched, and Subdrilled Holes

After shop assembly and before reaming, all punched, subpunched, and subdrilled holes shall meet the following standard of accuracy. At least 75 percent of the holes in each connection shall permit the passage of a cylindrical pin ⅛ inch smaller in diameter than nominal hole size. This pin shall pass through at right angles to the face of the member without drifting. All holes shall permit passage of a pin 3⁄16 inch smaller in diameter than nominal hole size. The Contracting Agency will reject any pieces that fail to meet these standards.

6-03.3(27)E Accuracy of Reamed and Drilled Holes

At least 85 percent of all holes in a connection of reamed or drilled holes shall show no offset greater than 1⁄32 inch between adjacent thicknesses of metal. No hole shall have an offset greater than 1⁄16 inch.

Centerlines from the connection shall be inscribed on the template and holes shall be located from these centerlines. Centerlines shall also be used for accurately locating the template relative to the milled or scribed ends of the members.

Templates shall have hardened steel bushing inserted into each hole. These bushings may be omitted, however, if the fabricator satisfies the Engineer (1) that the template will be used no more than five times, and (2) that use will produce no template wear.

Each template shall be at least ½ inch thick. If necessary, thicker templates shall be used to prevent buckling and misalignment as holes are formed.

6-03.3(27)F Fitting for Bolting

Before drilling, reaming, and bolting begins, all parts of a member shall be assembled, well pinned, and drawn firmly together. If necessary, assembled pieces shall be taken apart to permit removal of any burrs or shavings produced as the holes are formed. The member shall be free from twists, bends, and other deformation.

In shop-bolted connections, contacting metal surfaces shall be sandblasted clean before assembly. Sandblasting shall meet the requirements of the SSPC Specifications for Commercial Blast Cleaning (SSPC-SP 6).

Any drifting done during assembly shall be no more than enough to bring the parts into place. Drifting shall not enlarge the holes or distort the metal.

6-03.3(28) Shop Assembly

6-03.3(28)A Method of Shop Assembly

Unless the Contract states otherwise, the Contractor shall choose one of the five shop assembly methods described below that will best fit the proposed erection method. The Contractor shall obtain the Engineer’s approval of both the shop assembly and the erection methods before Work begins.

1. **Full Truss or Girder Assembly** – Each truss or girder is completely assembled over the full length of the Superstructure.
2. **Progressive Truss or Girder Assembly** – Each truss or girder is assembled in stages longitudinally over the full length of the Superstructure.
    a. **For Trusses** – The first stage shall include at least three adjacent truss panels. Each truss panel shall include all of the truss members in the space bounded by the top and bottom chords and the horizontal distance between adjacent bottom chord Joints.
    b. **For Girders** – The first stage shall include at least three adjacent girder shop sections. Shop sections are measured from the end of the girder to the first field splice or from field splice to field splice.
    c. **For Trusses and Girders** – After the first stage has been completed, each subsequent stage shall be assembled to include: at least one truss panel or girder shop section of the previous stage and two or more truss panels or girder shop sections added at the advancing end. The previous stages shall be repositioned if necessary, and pinned to ensure accurate alignment. For straight sections of bridges without skews or tapers, girders in each subsequent stage may be assembled to include one girder shop section from the previous stage and one or more girder shop sections at the advancing end.

        If the bridge is longer than 150 feet, each longitudinal stage shall be at least 150 feet long, regardless of the length of individual continuous truss panels or girder shop sections.

        The Contractor may begin the assembly sequence at any point on the bridge and proceed in either or both directions from that point.

        Unless the Engineer approves otherwise, no assembly shall have less than three truss panels or girder shop sections.
3. **Full Chord Assembly** – The full length of each chord for each truss is assembled with geometric angles at the joints. Chord connection bolt holes are drilled/reamed while members are assembled. The truss web member connections are drilled/reamed to steel templates set by relating geometric angles to the chord lines.

    At least one end of each web member shall be milled or scribed at right angles to its long axis. The templates at both ends of the member shall be positioned accurately from the milled end or scribed line.

4. **Progressive Chord Assembly** – Adjacent chord sections are assembled in the same way as specified for Full Chord Assembly, using the procedure specified for Progressive Truss or Girder Assembly.
5. **Special Complete Structure Assembly** – All structural steel members (Superstructure and Substructure, including all secondary members) are assembled at one time.

6-03.3(28)B Check of Shop Assembly

The Contractor shall check each assembly for alignment, accuracy of holes, fit of milled joints, and other assembly techniques. Drilling or reaming shall not begin until the Engineer has given approval. If the Contractor uses N/C drilling, this approval must be obtained before the assembly or stage is dismantled.

6-03.3(29) Welded Shear Connectors

Installation, production control, and inspection of welded shear connectors shall conform to Chapter 9 of the AASHTO/AWS D1.5M/D1.5, latest edition, Bridge Welding Code. If welded shear connectors are installed in the shop, installation shall be completed prior to applying the shop primer coat in accordance with Section 6-07.3(9)G. If welded shear connectors are installed in the field, the steel surface to be welded shall be prepared to SSPC-SP 11, power tool cleaning, just prior to welding.

6-03.3(30) Painting

All painting shall be in accordance with Section 6-07.

6-03.3(30)A Vacant

6-03.3(30)B Vacant

6-03.3(30)C Erection Marks

Erection marks to permit identification of members in the field shall be painted on previously painted surfaces.

6-03.3(30)D Machine Finished Surfaces

As soon as possible and before they leave the shop, machine-finished surfaces on abutting chord splices, column splices, and column bases shall be covered with grease. After erection, the steel shall be cleaned and painted as specified.

All surfaces of iron and steel castings milled to smooth the surface shall be painted with the primer called for in the specified paint system.

While still in the shop, machine-finished surfaces and inaccessible surfaces of rocker or pin-type bearings shall receive the full paint system. Surfaces of pins and holes machine-finished to specific tolerances shall not be painted. But as soon as possible and before they leave the shop, they shall be coated with grease.

6-03.3(31) Alignment and Camber

Before beginning field bolting, the Contractor shall:

1. Adjust the Structure to correct grade and alignment,
2. Regulate elevations of panel points (ends of floorbeams), and
3. Delay bolting at compression joints until adjusting the blocking to provide full and even bearing over the whole joint.

On truss spans, a slight excess camber will be permitted as the bottom chords are bolted. But camber and relative elevations of panel points shall be correct before the top chord joints, top lateral system, and sway braces are bolted.

6-03.3(31)A Measuring Camber

The Contractor shall provide the Engineer with a diagram for each truss that shows camber at each panel point. This diagram shall display actual measurements taken as the truss is being assembled.

6-03.3(32) Assembling and Bolting

To begin bolting any field connection or splice, the Contractor shall install and tighten to snug tight enough bolts to bring all parts into full contact with each other prior to tightening these bolts to the specified minimum tension. "Snug tight" means either the tightness reached by (l) a few blows from an impact wrench or (2) the full effort of a person using a spud wrench.

As erection proceeds, all field connections and splices for each member shall be securely drift pinned and bolted in accordance with 1 or 2 below before the weight of the member can be released or the next member is added. Field erection drawings shall specify pinning and bolting requirements that meet or exceed the following minimums:

1. **Joints in Normal Structures** – Fifty percent of the holes in a single field connection and 50 percent of the holes on each side of a single joint in a splice plate shall be filled with drift pins and bolts. Thirty percent of the filled holes shall be pinned. Seventy percent of the filled holes shall be bolted and tightened to snug tight. Once all these bolts are snug tight, each bolt shall be systematically tightened to the specified minimum tension. "Systematically tightened" means beginning with bolts in the most rigid part, which is usually the center of the joint, and working out to its free edges. The fully tensioned bolts shall be located near the middle of a single field connection or a single splice plate.
2. **Joints in Cantilevered Structures** – Seventy-five percent of the holes in a single field connection and 75 percent of the holes on each side of a single joint in a splice plate shall be filled with drift pins and bolts. Fifty percent of the filled holes shall be pinned. Fifty percent of the filled holes shall be bolted and tightened to snug tight. Once all these bolts are snug tight, each bolt shall be systematically tightened to the specified minimum tension. The fully tensioned bolts shall be located near the middle of a single field connection or a single splice plate.

Cylindrical erection pins (drift pins) shall be placed throughout each field connection and each field joint with the greatest concentration in the outer edges of a splice plate or member being bolted. Drift pins shall be double-tapered barrel pins of hardened steel. The diameter of the drift pins shall be at least $\frac{1}{32}$ inch larger than the diameter of the bolts in the connection or the full hole diameter.

To complete a joint following one of the methods listed above, the Contractor shall fill all remaining holes of the field connection or splice plate with bolts and tighten to snug tight. Once all of these bolts are snug tight, each bolt shall be systematically tightened to the specified minimum tension. After these bolts are tightened to the specified minimum tension, the Contractor shall replace the drift pins with bolts tightened to the specified minimum tension.

The Contractor shall complete the joint or connection within ten calendar days of installing the first bolt or within a duration approved by the Engineer. Any bolts inserted in an incomplete connection, either loose or tightened snug-tight, which exceed the specified duration for completing the connection, shall be subject to the following requirements:

1. Three assemblies for each size and length shall be removed from connection(s) that are to be tensioned. Rotational capacity tests shall be performed on the removed assemblies to demonstrate the assembly has sufficient lubricant to be tensioned satisfactorily.
2. Five assemblies shall be removed from the connection to establish the inspection torque.

3. In the case of tension controlled bolts, three assemblies shall be removed and tested in accordance with Section 6-03.3(33)A to verify the minimum specified tension can be achieved prior to shearing of the spline.

Assemblies removed for the purpose of rotational capacity testing, determination of the inspection torques, or verification of tension controlled bolt performance shall be replaced with new bolts at no additional expense to the Contracting Agency. To minimize the number of removed assemblies, the Contractor may combine rotational capacity testing and inspection torque determination as approved by the Engineer.

The Contractor may complete a field bolted connection or splice in a continuous operation before releasing the mass of the member or adding the next member. The Contractor shall utilize drift pins to align the connection. The alignment drift pins shall fill between 15 and 30 percent of the holes in a single field connection and between 15 and 30 percent of the holes on each side of a single joint in a splice plate. Once the alignment drift pins are in place, all remaining holes shall be filled with bolts and tightened to snug tight starting from near the middle and proceeding toward the outer gage lines. Once all of these bolts are snug tight, the Contractor shall systematically tighten all these bolts to the specified minimum tension. The Contractor shall then replace the drift pins with bolts. Each of these bolts shall be tightened to the specified minimum tension.

All bolts shall be placed with heads toward the outside and underside of the bridge. All high-strength bolts shall be installed and tightened before the falsework is removed.

The Contractor may erect metal railings as erection proceeds. But railings shall not be bolted or adjusted permanently until the falsework is released and the deck placed.

The Contractor shall not begin painting until the Engineer has inspected and accepted field bolting.

6-03.3(33) Bolted Connections

Fastener components shall consist of bolts, nuts, washers, tension control bolt assemblies, and direct tension indicators. Fastener components shall meet the requirements of Section 9-06.5(3). After final tightening of the fastener components, the threads of the bolts shall at a minimum be flush with the end of the nut.

The Contractor shall submit Type 1 Working Drawings providing documentation of the bolt tension calibrator, including brand, capacity, model, date of last calibration, and manufacturer's instructions for use. The Contractor shall supply the bolt tension calibrator and all accompanying hardware and calibrated torque wrenches to conduct all testing and inspections described herein. Use of the bolt tension calibrator shall comply with manufacturer's recommendations.

Fastener components shall be protected from dirt and moisture in closed containers at the site of installation. Only as many fastener components as are anticipated to be installed during the Work shift shall be taken from protected storage. Fastener components that are not incorporated into the Work shall be returned to protected storage at the end of the Work shift. Fastener components shall not be cleaned or modified from the as-delivered condition. Fastener components that accumulate rust or dirt shall not be incorporated into the Work. Tension control bolt assemblies shall not be relubricated, except by the manufacturer.

All bolted connections are slip critical unless otherwise noted in the Plans. Bolts installed in the field for painted structures require either Type 1 or Type 3 bolts and shall be galvanized in accordance with ASTM F2329 or ASTM B695 Class 55. All components of the fastener assembly including bolt, nut and washer, shall be galvanized using the same method. When galvanized bolts are specified, tension-control galvanized bolts are not permitted. Unpainted structures require Type 3 bolts. ASTM F3125 Grade A490 bolts shall not be galvanized and shall not be used in contact with galvanized metal.

Washers are required under turned elements for bolted connections and as required in the following:

1. Washers shall be used under both the head and the nut when ASTM F3125 Grade A490 bolts are to be installed in structural carbon steel, as specified in Section 9-06.1.
2. Where the outer face of the bolted parts has a slope greater than 1:20 with respect to a plane normal to the bolt axis, a beveled washer shall be used.
3. Washers shall not be stacked unless otherwise specified by the Engineer.
4. It is acceptable to place a washer under the unturned element.

All galvanized nuts shall be lubricated by the manufacturer with a lubricant containing a visible dye so a visual check for the lubricant can be made at the time of field installation. Black bolts shall be lubricated by the manufacturer and shall be “oily” to the touch when installed.

After assembly, bolted parts shall fit solidly together. They shall not be separated by washers, gaskets, or any other material. Assembled joint surfaces, including those next to bolt heads, nuts, and washers, shall be free of loose mill scale, burrs, dirt, and other foreign material that would prevent solid seating.

When all bolts in a joint are tight, each bolt shall carry at least the proof load shown in Table 1 below:

Table 1 Minimum Bolt Tension

Bolt Size (inches)	ASTM F3125 Grade A325 and Grade F1852 (pounds)	ASTM F3125 Grade A490 (pounds)
½	12,050	17,050
⅝	19,200	27,100
¾	28,400	40,100
⅞	39,250	55,450
1	51,500	72,700
1⅛	64,900	91,550
1¼	82,400	116,300
1⅜	98,200	138,600
1½	119,500	168,600

Prior to final tightening of any bolts in a bolted connection, the connection shall be compacted to a snug tight condition. Snug tight shall include bringing all plies of the connection into firm contact and snug tightening all bolts in accordance with Section 6-03.3(32).

Final tightening may be done by the Turn-of-Nut Method, the direct-tension indicator method, or the twist off-type tension control structural bolt/nut/washer assembly method. Preferably, the nut shall be turned tight while the bolt is prevented from rotating. However, if required by either turn-of-nut or direct-tension-indicator methods because of bolt entering and/or wrench operational clearances, tightening may be done by turning the bolt while the nut is prevented from rotating.

1. **Turn-of-Nut Method** – After all specified bolting conditions are satisfied, and before final tightening, the Contractor shall match-mark with crayon or paint the outer face of each nut and the protruding part of the bolt. Each bolt shall be final tightened to the specified minimum tension by rotating the amount specified in Table 2. To ensure this tightening method is followed, the Engineer will (1) observe as the Contractor installs, snug tightens, and final tightens all bolts and (2) inspect each match-mark.

Table 2 Turn-of-Nut Tightening Method Nut Rotational From Snug Tight Condition

Bolt Length	Disposition of Outer Faces of Bolted Parts		
	Condition 1	Condition 2	Condition 3
L <= 4D	⅓-turn	½-turn	⅔-turn
4D < L<= 8D	½-turn (*)	⅔-turn	⅚-turn
8D < L<= 12D	⅔-turn (*)	⅚-turn	1-turn

(*) When performing rotational capacity testing in accordance with Section 6-03.3(33)A, the total of two times the turns specified in Table 2 may be reduced by 15% for ASTM F3125 GR A490 bolts when allowed by the Engineer.

Bolt length measured from underside of head to top of nut.

Condition 1 – Both faces at right angles to bolt axis.

Condition 2 – One face at right angle to bolt axis, one face sloped no more than 1:20, without bevel washer.

Condition 3 – Both faces sloped no more than 1:20 from right angle to bolt axis, without bevel washer.

Nut rotation is relative to the bolt regardless of which element (nut or bolt) is being turned. Tolerances permitted plus or minus 30 degrees (1⁄12-turn) for final turns of ½-turn or less; plus or minus 45 degrees (⅛-turn) for final turns of ⅔-turn or more.

D = nominal bolt diameter of bolt being tightened.

When bolt length exceeds 12D, the rotation shall be determined by actual tests in which a suitable tension device simulates actual conditions.

2. **Direct Tension Indicator Method (DTIs)**. DTIs shall be placed under the bolt head with the protrusions facing the bolt head when the nut is turned. DTIs shall be placed under the nut with the protrusions facing the nut when the bolt is turned. DTIs may be placed under the turned element only if a hardened washer is used to separate the turned element from the DTI.

   Gap refusal shall be measured with a 0.005 inch tapered feeler gage. After all specified bolting conditions are satisfied, the snug tightened gaps shall meet Table 3 snug tight limits.

   Each bolt shall be final-tightened to meet Table 3 final-tighten limits. If the bolt is tensioned so that no visible gap in any space remains, the bolt and DTI shall be removed and replaced by a new properly tensioned bolt and DTI.

   The Contractor shall tension all bolts, inspecting all DTIs with a feeler gage, in the presence of the Engineer. DTIs shall be installed by two-person (or more) crews, with one individual (1) preventing the element at the DTI from turning and (2) measuring the gap of the DTI to determine the proper tension of the bolt.

   If a bolt, that has had its DTI brought to full load, loosens during the course of bolting the connection, it shall be rejected. Reuse of the bolt and nut are subject to the provisions of this section. The used DTI shall not be reinstalled.

Table 3 Direct Tension Indicator Requirements

Bolt Size (inches)	DTI Spaces		Maximum Snug Tight Refusals		Minimum Final Tighten Refusals	
	ASTM F3125 Grade A 325	ASTM F3125 Grade A490	ASTM F3125 Grade A 325	ASTM F3125 Grade A490	ASTM F3125 Grade A 325	ASTM F3125 Grade A490
½	4	5	1	2	2	3
⅝	4	5	1	2	2	3
¾	5	6	2	2	3	3
⅞	5	6	2	2	3	3
1	6	7	2	3	3	4
1⅛	6	7	2	3	3	4
1¼	7	8	3	3	4	4
1⅜	7	8	3	3	4	4
1½	8	9	3	4	4	5

3. **Twist Off-Type Tension Control Structural Bolt/Nut/Washer Assembly Method (Tension Control Bolt Assembly)** – Tension control bolt assemblies shall include the bolt, nut, and washer(s) packaged and shipped as a single assembly. Unless otherwise accepted by the Engineer, tension control bolt assembly components shall not be interchanged for testing or installation and shall comply with all provisions of ASTM F3125 Grade F1852. If accepted by the Engineer, the tension control bolt assembly components may be interchanged within the same component lot for girder web splices or other locations where access to both sides of the connection is restricted.

   The tension control bolts shall incorporate a design feature intended to either indirectly indicate, or to automatically provide, the minimum tension specified in Table 1.

   The Contractor shall submit Type 1 Working Drawings of the tension control bolt assembly, including bolt capacities; type of bolt, nut, and washer lubricant; method of packaging and protection of the lubricated bolt; installation equipment; calibration equipment; and installation procedures.

   The tension control bolt manufacturer's installation procedure shall be followed for installation of bolts in the verification testing device, in all calibration devices, and in all structure connections.

   In some cases, proper tensioning of the bolts may require more than one cycle of systematic partial tightening prior to final yield or fracture of the tension control element of each bolt. If yield or fracture of the tension control element of a bolt occurs prior to the final tightening cycle, that bolt shall be replaced with a new one.

   Additional field verification testing shall be performed as requested by the Engineer.

   All bolts and connecting hardware shall be stored and handled in a manner to prevent corrosion and loss of lubricant. Bolts that are installed without the same lubricant coating as tested under the verification test will be rejected, and they shall be removed from the joint and be replaced with new lubricated bolts at no additional cost to the Contracting Agency.

ASTM F3125 Grade A490 bolts, galvanized ASTM F3125 Grade A325 bolts, and ASTM F3125 Grade F1852 tension control bolt assemblies shall not be reused. Black ASTM F3125 Grade A325 bolts may be reused once if accepted by the Engineer. All bolts to be reused shall have their threads inspected for distortion by reinstalling the used nut on the bolt and turning the nut for the full length of the bolt threads by hand. Bolts to be reused shall be relubricated in accordance with the manufacturer's recommendation. Used bolts shall be subject to a rotational capacity test as specified in Section 6-03.3(33)A Pre-Erection Testing. Touching up or retightening bolts previously tightened by the Turn-of-Nut Method, which may have been loosened by the tightening of adjacent bolts shall not be considered as reuse, provided the snugging up continues from the initial position and does not require greater rotation, including the tolerance, than that required by Table 2.

6-03.3(33)A Pre-Erection Testing

High-strength bolt assemblies (bolt, nut, direct tension indicator, and washer), both black and galvanized, shall be subjected to a field rotational capacity test, as outlined below, prior to any permanent fastener installation. For field installations, the rotational capacity test shall be conducted at the jobsite. Each combination of bolt production lot, nut production lot, washer production lot, and direct tension indicator production lot shall be tested as an assembly, except tension control bolt assemblies, which shall be tested as supplied by the manufacturer. Each rotational capacity test shall include three assemblies. Once an assembly passes the rotational capacity test, it is accepted for use for the remainder of the project unless the Engineer deems further testing is necessary. All tests shall be performed in a bolt tension calibrator by the Contractor in the presence of the Engineer. High-strength bolt assemblies used in this test shall not be reused. The bolt assemblies shall meet the following requirements after being pretensioned to 15 percent of the minimum bolt tension in Table 1. The assembly shall be considered as nonconforming if the assembly fails to pass any one of the following specified requirements:

1. The measured torque to produce the minimum bolt tension shall not exceed the maximum allowed torque value obtained by the following equation:

   $$\text{Torque} = 0.25\ PD$$

   Where:

   Torque = Calculated Torque (foot-pounds)
   P = Measured Bolt Tension (pounds)
   D = Normal Bolt Diameter (feet)

2. After placing the assembly through two times the required number of turns, where turns are measured from the 15 percent pretension condition, as indicated in Table 2,

   a. The maximum recorded tension after the two turns shall be equal to or greater than 1.15 times the minimum bolt tension listed in Table 1.

   b. Each assembly shall be successfully installed to the specified number of turns.

   c. The fastener components in the assembly shall not exhibit shear failure or stripping of the threads as determined by visual examination of bolt and nut threads following removal.

   d. The bolts in the assembly shall not exhibit torsional or torsional/tension failure.

3. If any specimen fails, the assembly will be rejected. Elongation of the bolt between the bolt head and the nut is not considered to be a failure.

Bolts that are too short to test in the bolt tension calibrator shall be tested in a steel joint. The Contractor shall (1) install the high-strength bolt assemblies (bolt, nut, direct tension indicator, and washer) in a steel joint of the proper thickness; (2) tighten to the snug tight condition; (3) match-mark the outer face of each nut and the protruding part of the bolt with crayon or paint; (4) rotate to the requirements of Table 2; and (5) record the torque that is required to achieve the required amount of rotation. The assembly shall be considered as nonconforming if the assembly fails to pass any one of the following specified requirements:

1. The recorded torque to produce the minimum rotation shall not exceed the maximum allowed torque value obtained by the following equation:

   $$\text{Torque} = 0.25\ PD$$

   Where:

Torque	=	Calculated Maximum Allowed Torque (foot-pounds)
P	=	Specified Bolt Tension per Table 1, multiplied by a factor of 1.15 (pounds)
D	=	Normal Bolt Diameter (feet)

2. After placing the assembly through two times the required number of turns, where turns are measured from the snug tight condition specified in Section 6-03.3(32):
   a. Each assembly shall be successfully installed to the specified number of turns.
   b. The fastener components in the assembly shall not exhibit shear failure or stripping of the threads as determined by visual examination of bolt and nut threads following removal.
   c. The bolts in the assembly shall not exhibit torsional or torsional/tension failure.
3. If any specimen fails, the assembly will be rejected. Elongation of the bolt between the bolt head and the nut is not considered to be a failure.

The Contractor shall submit Type 1 Working Drawings consisting of the manufacturer's detailed procedure for pre-erection (rotational capacity) testing of tension control bolt assemblies.

Three DTIs, per lot, shall be tested in a bolt tension calibrator. The bolts shall be tensioned to 105 percent of the tension shown in Table 1. If all of the DTI protrusions are completely crushed (all five openings with zero gap), this lot of DTIs is rejected.

Three twist off-type tension controlled bolt assemblies, per assembly lot, shall be tested in a bolt tension calibrator. The bolts shall first be tensioned to a snug tight condition. Tensioning shall then be completed by tightening the assembly nut in a continuous operation using a spline drive installation tool until the spline shears from the bolt. The bolt assembly tension shall meet the requirements of Table 1. If any specimen fails, the assembly lot is rejected.

6-03.3(33)B Bolting Inspection

The Contractor, in the presence of the Engineer, shall inspect the tightened bolt using a calibrated inspection torque wrench, regardless of bolting method. The Contractor shall supply the inspection torque wrench. Inspection shall be performed within seven calendar days from the completion of each bolted connection or as specified by the Engineer.

If the bolts to be installed are not long enough to fit in the bolt tension calibrator, five bolts of the same grade, size, and condition as those under inspection shall be tested using Direct-Tension-Indicators (DTIs) to measure bolt tension. This tension measurement test shall be done at least once each inspection day. The Contractor shall supply the necessary DTIs. The DTI shall be placed under the bolt head. A washer shall be placed under the nut, which shall be the element turned during the performance of this tension measurement test. Each bolt shall be tightened by any convenient means to the specified minimum tension as indicated by the DTI. The inspecting wrench shall then be applied to the tightened bolt to determine the torque required to turn the nut 5 degrees (approximately 1 inch at a 12-inch radius) in the tightening direction. The job inspection torque shall be taken as the average of three values thus determined after rejecting the high and low values.

Five representative bolts/nuts/washers and DTIs if used (provided by the Contractor) of the same grade, size, and condition as those under inspection shall be placed individually in a bolt tension calibrator to measure bolt tension. This calibration operation shall be done at least once each inspection day. There shall be a washer under the part turned in tightening each bolt if washers are used on the Structure. In the bolt tension calibrator, each bolt shall be tightened by any convenient means to the specified tension. The inspection torque wrench shall then be applied to the tightened bolt to determine the torque required to turn the nut or head 5 degrees (approximately 1 inch at a 12-inch radius) in the tightening direction. The job-inspection torque shall be taken as the average of three values thus determined after rejecting the high and low values.

Ten percent (at least two), or as specified by the Engineer, of the tightened bolts on the Structure represented by the test bolts shall be selected at random in each connection. The job-inspection torque shall then be applied to each with the inspecting wrench turned in the tightening direction, with no restraint applied to the opposite end of the bolt. If this torque turns no bolt head or nut, the Contracting Agency will accept the connection as being properly tightened. If the torque turns one or more bolt heads or nuts, the job-inspection torque shall then be applied to all bolts in the connection. Except for tension control bolt assemblies and DTIs with zero gap at all protrusion spaces, any bolt whose head or nut turns at this stage shall be tightened and reinspected. Any tension control bolt assemblies or DTIs that have zero gap at all protrusion spaces shall be replaced if the head or nut turns at this stage.

The Contractor shall submit Type 1 Working Drawings consisting of the manufacturer's detailed procedure for routine observation to ensure proper use of the tension control bolt assemblies.

6-03.3(34) Adjusting Pin Nuts

All pin nuts shall be tightened thoroughly. The pins shall be placed so that members bear fully and evenly on the nuts. The pins shall have enough thread to allow burring after the nuts are tightened.

6-03.3(35) Setting Anchor Bolts

Anchor bolts shall be set in masonry as required in Section 6-02.3(18). Anchor bolts shall be grouted in after the shoes, masonry plates, and keeper plates have been set and the span or series of continuous spans are completely erected and adjusted to line and camber.

6-03.3(36) Setting and Grouting Masonry Plates

The following procedure applies to masonry plates for all steel spans, including shoes, keeper plates, and turning racks on movable bridges.

To set masonry plates, the Contractor shall:

1. Set masonry plates on the anchor bolts;
2. Place steel shims under the masonry plates to position pin centers or bearings to line and grade and in relationship to each other. Steel shims shall be the size and be placed at the locations shown in the Plans;
3. Level the bases of all masonry plates;
4. Draw anchor bolt nuts down tight;
5. Recheck pin centers or bearings for alignment; and
6. Leave at least ¾ inch of space under each masonry plate for grout.

After the masonry plates have been set and the span or series of continuous spans are completely erected and swung free, the space between the top of the masonry and the top of the concrete bearing seat shall be filled with grout. Main masonry plates for cantilever spans shall be set and grouted in before any steel Work is erected.

Grout shall conform to Section 9-20.3(2) and placement shall be as required in Section 6-02.3(20).

6-03.3(37) Setting Steel Bridge Bearings

Masonry plates, shoes, and keeper plates of expansion bearings shall be set and adjusted to center at a normal temperature of 64°F. Adjustment for an inaccuracy in fabricated length shall be made after dead-load camber is out.

6-03.3(38) Placing Superstructure

The concrete in piers and crossbeams shall reach at least 80 percent of design strength before girders are placed on them.

6-03.3(39) Swinging the Span

Forms weighing less than 5 pounds per square foot of bridge deck area and uniformly distributed along the steel spans may be placed before the spans swing free on their supports. Steel reinforcing bars or concrete bridge deck shall not be placed on steel spans until the spans swing free on their supports and elevations are recorded. No simple span or any series of continuous spans will be considered as swinging free until all temporary supports have been released. Reinforcing steel or concrete bridge decks shall not be placed on any simple or continuous span steel girder bridge until all its spans are adjusted and its masonry plates, shoes, and keeper plates grouted. For this specification, the structure shall be considered as continuous across hinged joints.

After the falsework is released (spans swung free), the masonry plates, shoes, and keeper plates are grouted, and before any load is applied, the Contractor (or the Engineer if the Contracting Agency is responsible for surveying) shall survey elevations at the tenth points along the centerline on top of all girders and floorbeams. The Contractor shall calculate the theoretical top of girder or floorbeam flange elevations and compare the calculated elevations to the surveyed elevations. The theoretical pad or haunch depth shown in the Plans shall be increased or decreased by the difference between the theoretical and surveyed top of girder or floorbeam elevations. The soffit (deck formwork) shall be set based on the Plan bridge deck thickness and the adjusted pad or haunch depth.

The Contractor shall submit all survey data and calculations to the Engineer for review ten working days prior to placing any load, beyond the maximum five pounds per square foot of form weight allowed, on the Structure.

6-03.3(40) Draining Pockets

The Contractor shall provide enough holes to drain all water from pockets in trusses, girders, and other members. Unless shown on approved shop plans, drain holes shall not be drilled without the written approval of the Engineer.

All costs related to providing drain holes shall be included in the unit Contract prices for structural or cast steel.

6-03.3(41) Vacant

6-03.3(42) Surface Condition

As the Structure is erected, the Contractor shall keep all steel surfaces clean and free from dirt, concrete, mortar, oil, paint, grease, and other stain-producing foreign matter. Any surfaces that become stained shall be cleaned as follows:

> Painted steel surfaces shall be cleaned by methods required for the type of staining. The Contract shall submit a Type 1 Working Drawing of the cleaning method.
>
> Unpainted steel surfaces shall be cleaned by sandblasting. Sandblasting to remove stains on publicly visible surfaces shall be done to the extent that, in the Engineers opinion, the uniform weathering characteristics of the Structure are preserved.

6-03.3(43) Castings, Steel Forgings, and Miscellaneous Metals

Castings, steel forgings, and miscellaneous metals shall be built to comply with Section 9-06.

6-03.3(43)A Shop Construction, Castings, Steel Forgings, and Miscellaneous Metals

This section's requirements for structural steel (including painting requirements) shall also apply to castings, steel forgings, and miscellaneous metals.

Castings shall be:

1. True to pattern in form and dimensions;
2. Free from pouring faults, sponginess, cracks, blow holes, and other defects in places that would affect strength, appearance, or value;
3. Clean and uniform in appearance;
4. Filleted boldly at angles; and
5. Formed with sharp and perfect arises.

Iron and steel castings and forgings shall be annealed before any machining, unless the Plans state otherwise.

6-03.4 Measurement

Cast or forged metal (kind) shown in the Plans will be measured by the pound or will be paid for on a lump sum basis, whichever is shown on the Proposal.

6-03.5 Payment

Payment will be made for each of the following Bid items that are included in the Proposal:

"Structural Carbon Steel", lump sum.

The lump sum Contract price for "Structural Carbon Steel" shall be full pay for all costs in connection with furnishing all materials, labor, tools, and equipment necessary for the manufacture, fabrication, transportation, erection, and painting of all structural carbon steel used in the completed Structure, including the providing of such other protective coatings or treatment as may be shown in the Plans or specified in the Special Provisions.

For steel Structures, the estimated weight of the structural carbon steel in the project will be shown in the Plans or in the Special Provisions. In the event any change in the Plans is made which will affect the weight of materials to be furnished, payment for the additional structural carbon steel required as a result of the change in the Plans will be made at a unit price per pound obtained by dividing the Contractor's lump sum Bid for structural carbon steel by the total estimated weight of structural carbon steel shown in the Plans or in the Special Provisions.

Reductions in weight due to a change in the Plans will be made at the same rate as determined above and will be deducted from payments due the Contractor.

Prospective Bidders shall verify the estimated weight of structural carbon steel before submitting a Bid. No adjustment other than for approved changes shall be made in the lump sum Bid even though the actual weight may deviate from the stated estimated weight.

For concrete and timber Structures, where the structural carbon steel is a minor item, no estimated weight will be given for the structural carbon steel. In the event any change in the Plans is necessary which will affect the weight of material to be furnished for this type of Structure, the payment or reduction for the revision in quantity will be made at a unit price per pound obtained by dividing the Contractor's lump sum Bid for the structural carbon steel by the calculated weight of the original material. The calculated weight will be established by the Engineer and will be based on an estimated weight of 490 pounds per cubic foot for steel.

Any change in the Plans which affects the weight of material to be furnished as provided herein will be subject to the provisions of Section 1-04.4.

“Structural Low Alloy Steel”, lump sum.

“Structural High Strength Steel”, lump sum.

Payment for “Structural Low Alloy Steel” and “Structural High Strength Steel” shall be made on the same lump sum basis as specified for structural carbon steel.

“(Cast or Forged) Steel”, lump sum or per pound.

“(Cast, Malleable, or Ductile) Iron”, lump sum or per pound.

“Cast Bronze”, lump sum or per pound.

Payment for “(Cast or Forged) Steel”, “(Cast, Malleable or Ductile) Iron”, and “Cast Bronze” will be made at the lump sum or per pound Contract prices as included in the Proposal.

For the purpose of payment, such minor items as bearing plates, pedestals, forged steel pins, anchor bolts, field bolts, shear connectors, etc., unless otherwise provided, shall be considered as structural carbon steel even though made of other materials.

When no Bid item is included in the Proposal and payment is not otherwise provided, the castings, forgings, miscellaneous metal, and painting shall be considered as incidental to the construction, and all costs therefore shall be included in the unit Contract prices for the payment items involved and shown.

6-04 Timber Structures

6-04.1 Description

This Work is the building of any Structure or parts of Structures (except piling) made of treated timber, untreated timber, or both. The Contractor shall erect timber Structures on prepared foundations. The Structures shall conform to the dimensions, lines, and grades required by the Plans, the Engineer, and these Specifications.

Any part of a timber Structure made of nontimber materials shall comply with the sections of these Specifications that govern those materials.

6-04.2 Materials

Materials shall meet the requirements of the following sections:

Structural Steel and Related Material	9-06
Bolts, Washers, Other Hardware	9-06.22
Paints	9-08
Timber and Lumber	9-09

6-04.3 Construction Requirements

6-04.3(1) Storing and Handling Material

At the Work site, the Contractor shall store all timber and lumber in piles. Weeds and rubbish under and around these piles shall have been removed before the lumber is stacked.

Untreated lumber shall be open stacked at least 12 inches above the ground. It shall be piled to shed water and prevent warping.

Treated timber shall be:

1. Cut, framed, and bored (whenever possible) before treatment;
2. Close stacked and piled to prevent warping;
3. Covered against the weather if the Engineer requires it;
4. Handled carefully to avoid sudden drops, broken outer fibers, and surface penetration or bruising with tools; and
5. Lifted and moved with rope or chain slings (without use of cant dogs, peaveys, hooks, or pike poles).

6-04.3(2) Quality of Construction

The Contractor shall employ only competent bridge carpenters. All their Work shall be true and exact. Nails and spikes shall be driven with just enough force to leave heads flush with wood surfaces. The Contractor shall discharge any worker who leaves deep hammer marks in wood surfaces.

6-04.3(3) Shop Details

The Contractor shall submit Type 2 Working Drawings consisting of shop detail plans for all treated timber. These plans shall show dimensions for all cut, framed, or bored timbers.

6-04.3(4) Field Treatment of Cut Surfaces, Bolt Holes, and Contact Surfaces

All cut surfaces, bolt holes, and contact surfaces shall be treated in accordance with Section 9-09.3 for all timber and lumber requiring preservative treatment.

All cuts and abrasions in treated piles or timbers shall be trimmed carefully and treated in accordance with Section 9-09.3.

6-04.3(5) Holes for Bolts, Dowels, Rods, and Lag Screws

Holes shall be bored:

1. For drift pins and dowels – with a bit ¹⁄₁₆ inch smaller in diameter than the pins and dowels.
2. For truss rods or bolts – with a bit the same diameter as the rods or bolts.
3. For lag screws – in two parts: (a) with the shank lead hole the same diameter as the shank and as deep as the unthreaded shank is long; and (b) with the lead hole for the threaded part approximately ⅔ of the shank diameter.

6-04.3(6) Bolts, Washers, and Other Hardware

Bolts, dowels, washers, and other hardware, including nails, shall be black or galvanized as specified in the Plans, but if not so specified shall be galvanized when used in treated timber Structures.

Washers of the size and type specified shall be used under all bolt heads and nuts that would otherwise contact wood.

All bolts shall be checked by burring the threads after the nuts have been finally tightened. Vertical bolts shall have nuts on the lower ends.

Wherever bolts fasten timber to timber, to concrete, or to steel, the members shall be bolted tightly together at installation and retightened just before the Contracting Agency accepts the Work. These bolts shall have surplus threading of at least ⅜ inch per foot of timber thickness to permit future tightening.

6-04.3(7) Countersinking

Countersinking shall be done wherever smooth faces are required. Each recess shall be treated in accordance with Section 9-09.3.

6-04.3(8) Framing

The Contractor shall cut and frame lumber and timber to produce close-fitting, full-contact joints. Each mortise shall be true to size for its full depth, and its tenon shall fit it snugly. Neither shimmed nor open joints are permitted.

6-04.3(9) Framed Bents

Mudsills shall be of pressure-treated timber, firmly and evenly bedded to solid bearing, and tamped in place.

Concrete pedestals that support framed bents shall be finished so that sills will bear evenly on them. To anchor the sills, the Contractor shall set dowels in the pedestals when they are cast. The dowels shall be at least ¾ inch in diameter and protrude at least 6 inches above the pedestal tops. Pedestal concrete shall comply with Section 6-02.

Each sill shall rest squarely on mudsills, piles, or pedestals. It shall be drift-bolted to mudsills or piles with ¾-inch diameter or larger bolts that extend at least 6 inches into them. When possible, the Contractor shall remove any earth touching the sills to permit free air circulation around them.

Each post shall be fastened to sills with ¾-inch diameter or larger dowels that extend at least 6 inches into the post.

6-04.3(10) Caps

Timber caps shall rest uniformly across the tops of posts or piles and cap ends shall be aligned evenly. Each cap shall be fastened with a drift bolt ¾ inch in diameter or larger that penetrates the post or pile at least 9 inches. The bolt shall be approximately in the center of the pile or post.

If the Roadway grade exceeds 2 percent, each cap shall be beveled to match the grade.

6-04.3(11) Bracing

When pile bents are taller than 10 feet, each shall be braced transversely and every other pair shall be braced longitudinally. No single cross-bracing shall brace more than 20 feet of vertical distance on the piles. If the vertical distance exceeds 20 feet, more than one cross-bracing shall be used. Each brace end shall be bolted through the pile, post, or cap with a bolt ¾ inch in diameter or larger. Other brace/pile intersections shall be bolted or boat-spiked as the Plans require. Cross-bracing shall lap both upper or lower caps and shall be bolted to the caps or sills at each end.

6-04.3(12) Stringers

All stringers that carry laminated decking or vary more than ⅛ inch in depth shall be sized to an even depth at bearing points. Outside stringers shall be butt jointed and spliced. Interior stringers shall be lapped so that each rests over the full width of the cap or floorbeam at each end. Except on sharp horizontal and vertical curves, stringers may cover two spans. In this case, joints shall be staggered and the stringers either toenailed or drift bolted as the Plans require. To permit air circulation on untreated timber Structures, the ends of lapped stringers shall be separated. This separation shall be done by fastening across the lapping face a 1 by 3-inch wood strip cut 2 inches shorter than the depth of the stringer.

Any cross-bridging or solid bridging shall be neatly and accurately framed, then securely toenailed at each end (with two nails for cross-bridging and four nails for solid bridging). The Plans show bridging size and spacing.

6-04.3(13) Wheel Guards and Railings

Wheel guards and railings shall be built as Section 6-06.3(1) requires.

6-04.3(14) Single-Plank Floors

Single-plank floors shall be made of a single thickness of plank on stringers or joists. Unless the Engineer directs otherwise, the planks shall be:

1. Laid heart side down with tight joints,
2. Spiked to each joist or nailing strip with at least two spikes that are at least 4 inches longer than the plank thickness,
3. Spiked at least 2½ inches from the edges,
4. Cut off on a straight line parallel to the centerline of the Roadway,
5. Arranged so that no adjacent planks vary in thickness by no more than 1⁄16 inch, and
6. Surfaced on one side and one edge (S1S1E) unless otherwise specified.

6-04.3(15) Laminated Floors

The strips shall be placed on edge and shall be drawn down tightly against the stringer or nailing strip and the adjacent strip and, while held in place, shall be spiked. Each strip shall extend the full width of the deck, unless some other arrangement is shown in the Plans or permitted by the Engineer.

Each strip shall be spiked to the adjacent strip at intervals of not more than 2 feet, the spikes being staggered 8 inches in adjacent strips. The spikes shall be of sufficient length to pass through two strips and at least halfway through the third. In addition, unless bolting is specified in the Plans, each strip shall be toenailed to alternate stringers with 40d common nails and adjacent strips shall be nailed to every alternate stringer. The ends of all pieces shall be toenailed to the outside stringer. The ends of the strips shall be cut off on a true line parallel to the centerline of the Roadway. When bolts are used to fasten laminated floors to stringers, the bolts shall be placed at the spacing shown in the Plans, and the pieces shall be drawn down tightly to the bolting strips. The bolt heads shall be driven flush with the surface of the deck. Double nuts or single nuts and lock nuts shall be used on all bolts. The strips shall be spiked together in the same manner as specified above.

6-04.3(16) Plank Subfloors for Concrete Decks

Any plank subfloor shall be laid surfaced side down with close joints at right angles to the centerline of the Roadway. Planks shall be spiked in place as required in Section 6-04.3(14).

Floor planks shall be treated in accordance with Section 9-09.3.

6-04.3(17) Trusses

Completed trusses shall show no irregularities of line. From end to end, chords shall be straight and true in horizontal projection. In vertical projection they shall show a smooth curve through panel points that conforms to the correct camber. The Engineer will reject any pieces cut unevenly or roughly at bearing points. Before placement of the hand railing, the Contractor shall complete all trusses, swing them free of their falsework, and adjust them for line and camber (unless the Engineer directs otherwise).

6-04.3(18) Painting

Section 6-07.3(13) governs painting of timber Structures.

6-04.4 Measurement

The criteria in Section 6-03.4 will be used to determine the weight of structural metal other than hardware.

Timber and lumber (treated or untreated) will be measured by the 1,000-board feet (MBM), using nominal thicknesses and widths. Lengths will be actual lengths of individual pieces in the finished Structure with no deduction for daps, cuts, or splices. To measure laminated timber decking, the Contracting Agency will use the number and after-dressing sizes of pieces required in the Plans. The length of each lamination shall be the length remaining in the finished Structure.

6-04.5 Payment

Payment will be made for each of the following Bid items that are included in the Proposal:

1. “Timber and Lumber (untreated or name treatment)”, per MBM.
2. “Structural Metal”, lump sum.

Where no item for structural metal is included in the Proposal, full pay for furnishing and placing metal parts shall be included in the unit Contract price per MBM for “Timber and Lumber”.

When no Bid item is included in the Proposal and is not otherwise provided, painting shall be considered as incidental to the construction, and all costs therefore shall be included in the unit Contract prices for the payment items involved and shown.

6-05 Piling

6-05.1 Description

This Work consists of furnishing and driving piles (timber, precast concrete, cast-in-place concrete, and steel) of the sizes and types the Contract or the Engineer require. This Work also includes cutting off or building up piles when required. In furnishing and driving piles, the Contractor shall comply with the requirements of this section, the Contract, and the Engineer.

6-05.2 Materials

Materials shall meet the requirements of the following sections:

Reinforcing Steel	9-07
Prestressing Steel	9-07.10
Timber Piling	9-10.1
Concrete Piling	9-10.2
Cast-In-Place Concrete Piling	9-10.3
Steel Pile Tips and Shoes	9-10.4
Steel Piling	9-10.5
Mortar	9-20.4

6-05.3 Construction Requirements

6-05.3(1) Piling Terms

Concrete Piles – Concrete piling may be precast or precast-prestressed concrete, or steel casings driven to the nominal bearing resistance called for in the Contract which are filled with concrete (cast-in-place) after driving.

Steel Piles – Steel piles may be open-ended or closed-ended pipe piles, or H-piles.

Overdriving – Over-driving of piles occurs when the nominal bearing resistance calculated from the equation in Section 6-05.3(12), or the wave equation driving criteria if applicable, exceeds the nominal bearing resistance required in the Contract in order to reach the minimum tip elevation specified in the Contract, or as required by the Engineer.

Maximum Driving Resistance – The maximum driving resistance is either the pile nominal bearing resistance, or nominal bearing resistance plus overdriving to reach minimum tip elevation as specified in the Contract, whichever is greater.

Wave Equation Analysis – Wave equation analysis is an analysis performed using the wave equation analysis program (WEAP) with a version dated 1987 or later. The wave equation may be used as specified herein to verify the Contractor's proposed pile driving system. The pile driving system includes, but is not necessarily limited to, the pile, the hammer, the helmet, and cushions. The wave equation may also be used by the Engineer to determine pile driving criteria as may be required in the Contract.

Nominal Bearing Resistance – Nominal bearing resistance refers to the vertical load carrying resistance (in units of force) of a pile as determined by the equation in Section 6-05.3(12), the wave equation analysis, pile driving analyzer and CAPWAP, static load test, or any other means as may be required by the Contract, or the Engineer.

Rated Hammer Energy – The rated energy represents the theoretical maximum amount of gross energy that a pile driving hammer can generate. The rated energy of a pile driving hammer will be stated in the hammer manufacturer's catalog or Specifications for that pile driving hammer.

Developed Hammer Energy – The developed hammer energy is the actual amount of gross energy produced by the hammer for a given blow. This value will never exceed the rated hammer energy. The developed energy may be calculated as the ram weight times the drop (or stroke) for drop, single acting hydraulic, single acting air/steam, and open-ended diesel hammers. For double acting hydraulic and air/steam hammers, the developed hammer energy shall be calculated from ram impact velocity measurements or other means approved by the Engineer. For closed-ended diesel hammers, the developed energy shall be calculated from the measured bounce chamber pressure for a given blow. Hammer manufacturer calibration data may be used to correlate bounce chamber pressure to developed hammer energy. For a single acting diesel hammer the developed energy is determined using the blows per minute.

Transferred Hammer Energy – The transferred hammer energy is the amount of energy transferred to the pile for a given blow. This value will never exceed the developed hammer energy. Factors that cause transferred hammer energy to be lower than the developed hammer energy include friction during the ram down stroke, energy retained in the ram and helmet during rebound, and other impact losses. The transferred energy can only be measured directly by use of sensors attached to the pile. A pile driving analyzer (PDA) may be used to measure transferred energy.

Pile Driving Analyzer – A pile driving analyzer (PDA) is a device that can measure the transferred energy of a pile driving system, the compressive and tensile stresses induced in the pile due to driving, the bending stresses induced by hammer misalignment with the pile, and estimate the nominal bearing resistance of a pile at a given blow.

Pile Driving System – The pile driving system includes, but is not necessarily limited to, the hammer, leads, helmet or cap, cushion and pile.

Helmet – The helmet, also termed the cap, drive cap, or driving head, is used to transmit impact forces from the hammer ram to the pile top as uniformly as possible across the pile top such that the impact force of the ram is transmitted axially to the pile. The term helmet can refer to the complete impact force transfer system, which includes the anvil or striker plate, hammer cushion and cushion block, and a pile cushion if used, or just the single piece unit into which these other components fit. The helmet does not include a follower, if one is used. For hydraulic hammers, the helmet is sometimes referred to as the anvil.

Hammer Cushion – The hammer cushion is a disk of material placed on top of the helmet but below the anvil or striker plate to relieve impact shock, thus protecting the hammer and the pile.

Pile Cushion – The pile cushion is a disk of material placed between the helmet and the pile top to relieve impact shock, primarily to protect the pile.

Follower – A follower is a structural member placed between the hammer assembly, which includes the helmet, and the pile top when the pile head is below the reach of the hammer.

Pile Driving Refusal – Pile driving refusal is defined as 15 blows per inch for the last 4 inches of driving. This is the maximum blow count allowed during overdriving.

Minimum Tip Elevation – The minimum tip elevation is the elevation to which the pile tip shall be driven. Driving deeper in order to obtain the required nominal bearing resistance may be required.

6-05.3(2) Ordering Piling

The Contractor shall order all piling (except cast-in-place concrete and steel piles) from an order list the Engineer will provide. This list, showing the number and lengths of piles required, will be based on test-pile driving (or other) data. The order list will show lengths below the cutoff point. The Contractor shall supply (and bear the cost of supplying) all additional lengths required for handling or driving.

The Contractor shall assume all responsibility for buying more or longer piles than those shown on the list provided by the Engineer. All piles purchased on the basis of the Engineer's list but not used in the finished Structure shall become the property of the Contracting Agency. The Contractor shall deliver these as the Engineer directs. The pile cutoffs that are 8 feet or under and longer ones the Contracting Agency does not require shall become the property of the Contractor.

When ordering steel casings for cast-in-place concrete and steel piling, the Contractor shall base lengths on information derived from driving test piles and from subsurface data. The Contractor shall also select the wall thickness of steel piles or steel casings for cast-in-place piles which will be necessary to prevent damage during driving and handling. The selection of wall thickness for steel piles or steel casings shall also consider the effects of lateral pressures from the soil or due to driving of adjacent piles. Steel piles and steel casings must be strong and rigid enough to resist these pressures without deforming or distorting. The Contractor shall select the wall thickness based on information derived from test piles, subsurface data and/or wave equation analysis.

Wave equation analysis is required prior to ordering piling for piles with specified nominal bearing resistances of 300 tons or greater. If a wave equation analysis is performed, the Contractor shall base the selection of wall thickness on the maximum driving resistance identified in the Contract to reach the minimum tip elevation, if the maximum driving resistance is greater than the specified nominal bearing resistance and if a minimum tip elevation is specified. The wave equation analysis shall be submitted by the Contractor as required in Section 6-05.3(9)A. The Engineer will not supply any list for piling of these types.

6-05.3(3) Manufacture of Precast Concrete Piling

Precast concrete piles shall consist of concrete sections reinforced to withstand handling and driving stresses. These may be reinforced with deformed steel bars or prestressed with steel strands. The Plans show dimensions and details. If the Plans require piles with square cross-sections, the corners shall be chamfered 1 inch.

Precast or prestressed piles shall meet the requirements of the *Standard Plans*.

Temporary stress in the prestressing reinforcement of prestressed piles (before loss from creep and shrinkage) shall be 75 percent of the minimum ultimate tensile strength. (For short periods during manufacture, the reinforcement may be overstressed to 80 percent of ultimate tensile strength if stress after transfer to concrete does not exceed 75 percent of that strength.)

Prestressed concrete piles shall have a final (effective) prestress of at least 1,000 psi.

Unless the Engineer approves splices, all piles shall be full length.

The Contracting Agency intends to perform Quality Assurance Inspection. By its inspection, the Contracting Agency intends only to facilitate the Work and verify the quality of that Work. This inspection shall not relieve the Contractor of the responsibility for identifying and replacing defective material and Work.

6-05.3(3)A Casting and Stressing

Precast concrete piles shall be constructed in accordance with Section 6-02.3(9), except as modified in this section.

Reinforcing bars, hoops, and shoes shall be placed as shown in the Contract, with all parts securely tied together and placed to the specified spacing. No concrete shall be cast until all reinforcement is in place in the forms.

In casting concrete piles, the Contractor shall:

1. Cast them either vertically or horizontally;
2. Use metal forms (unless the Engineer approves otherwise) with smooth joints and inside surfaces that can be reached for cleaning after each use;
3. Brace and stiffen the forms to prevent distortion;
4. Place concrete continuously in each pile, guarding against horizontal or diagonal cleavage planes;
5. Ensure that the reinforcement is properly embedded;
6. Use internal vibration around the reinforcement during concrete placement to prevent rock pockets from forming; and
7. Cast test cylinders with each set of piles as concrete is placed.

Forms shall be metal and shall be braced and stiffened to retain their shape under pressure of wet concrete. Forms shall have smooth joints and inside surfaces easy to reach and clean after each use. That part of a form which will shape the end surface of the pile shall be a true plane at right angles to the pile axis.

Each pile shall contain a cage of nonprestressed reinforcing steel. The Contractor shall follow the Contract in the size and location of this cage and shall secure it in position during concrete placement. Spiral steel reinforcing shall be covered by at least 1½ inches of concrete measured from the outside pile surface.

Prestressing steel shall be tensioned as required in Section 6-02.3(25)C.

The Plans specify tensioning stress for strands or wires. Tension shall be measured by jack pressure as described in Section 6-02.3(25)C. Mechanical locks or anchors shall temporarily maintain cable tension. All jacks shall have hydraulic pressure gauges (accurately calibrated and accompanied by a certified calibration curve no more than 180 days old) that will permit stress calculations at all times.

All tensioned piles shall be pretensioned. Post-tensioning is not allowed.

The Contractor shall not stress piles until test cylinders made with it reach a compressive strength of at least 3,300 psi.

6-05.3(3)B Finishing

As soon as the forms for precast concrete piles are removed, the Contractor shall fill all holes and irregularities with mortar conforming to Section 9-20.4(2) mixed at a 1:2 cement/aggregate ratio. That part of a pile that will be underground or below the low-water line and all parts of a pile to be used in salt water or alkaline soil shall receive only this mortar treatment. That part of a pile that will show above the ground or water line shall be given a Class 2 finish as described in Section 6-02.3(14)B.

6-05.3(3)C Curing

Precast Concrete Piles – The Contractor:

1. Shall keep the concrete continuously wet with water after placement for at least 10 days with Type I or II portland cement or at least 3 days with Type III.
2. Shall remove side forms no sooner than 24 hours after concrete placement, and then only if the surrounding air remains at no less than 50°F for 5 days with Type I or II portland cement or 3 days with Type III.
3. May cure precast piles with saturated steam or hot air, as described in Section 6-02.3(25)D, provided the piles are kept continuously wet until the concrete has reached a compressive strength of 3,300 psi.

Precast-Prestressed Concrete Piles – These piles shall be cured as required in Section 6-02.3(25)D.

6-05.3(4) Manufacture of Steel Casings for Cast-In-Place Concrete Piles

The diameter of steel casings shall be as specified in the Contract. A full-penetration groove weld between welded edges is required.

6-05.3(5) Manufacture of Steel Piles

Steel piles shall be made of rolled steel H-pile sections, steel pipe piles, or of other structural steel sections described in the Contract. A full penetration groove weld between welded edges is required.

At least 14-days prior to the start of production of the piling, the Contractor shall advise the Engineer of the production schedule. The Contractor shall give the Inspector safe and free access to the Work. If the Inspector observes nonconforming Work or unacceptable quality control practices, the Inspector will advise the plant manager. If the corrective action is not acceptable to the Engineer, the piling(s) will be subject to rejection by the Engineer.

6-05.3(6) Splicing Steel Casings and Steel Piles

The Engineer will normally permit steel piles and steel casings for cast-in-place concrete piles to be spliced. But in each case, the Contractor shall submit Type 2 Working Drawings supporting the need and describing the method for splicing. Welded splices shall be spaced at a minimum distance of 10 feet. Only welded splices will be permitted.

Splice welds for steel piles shall comply with Section 6-03.3(25) and AWS D1.1/D1.1M, latest edition, Structural Welding Code. Splicing of steel piles shall be performed in accordance with an approved weld procedure. The Contractor shall submit a Type 2 Working Drawing consisting of the weld procedure. For ASTM A252 material, mill certification for each lot of pipe to be welded shall accompany the submittal. The ends of all steel pipe piling shall meet the fit-up requirements of AWS D1.1/D1.1M, latest edition, Structural Welding Code Section 5.22.3.1, “Girth Weld Alignment (Tubular),” when the material is spliced utilizing a girth weld.

Splice welds of steel casings for cast-in-place concrete piles shall be the Contractor’s responsibility and shall be welded in accordance with AWS D1.1/D1.1M, latest edition, Structural Welding Code. A weld procedure submittal is not required for steel casings used for cast-in-place concrete piles. Casings that collapse or are not watertight, shall be replaced at the Contractor’s expense.

6-05.3(7) Storage and Handling

The Contractor shall store and handle piles in ways that protect them from damage.

6-05.3(7)A Timber Piles

Timber piling shall be stacked closely and in a manner to prevent warping. The ground beneath and around stored piles shall be cleared of weeds, brush, and rubbish. Piling shall be covered against the weather if the Engineer requires it.

The Contractor shall take special care to avoid breaking the surface of treated piles. They shall be lifted and moved with equipment, tools, and lifting devices which do not penetrate or damage the piles. If timber piles are rafted, all attachments shall be within 3 feet of the butts or tips. All surfaces cut or broken shall be repaired in accordance with Section 9-09.3. The Engineer may reject piles because of a cut or break.

6-05.3(7)B Precast Concrete Piles

The Contractor shall not handle piles until test cylinders made with the same batch of concrete as the pile reach a compressive strength of at least 3,300 psi.

Storing and handling methods shall protect piles from fractures by impact and undue bending stresses. Handling methods shall never stress the reinforcement more than 12,000 psi. An allowance of twice the calculated load shall be made for impact and shock effects. The Contractor shall submit Type 2 Working Drawings consisting of the method of lifting the piles. The Contractor will take extra care to avoid damaging the surface of piles to be used in seawater or alkaline soil.

6-05.3(7)C Steel Casings and Steel Piles

The Engineer will reject bent, deformed, or kinked piles that cannot be straightened without damaging the metal.

6-05.3(8) Pile Tips and Shoes

The Contracting Agency prefers that timber piles be driven with squared ends. But if conditions require, they may be shod with metal shoes. Pile tips and shoes shall be securely attached to the piles in accordance with the manufacturer's recommendations.

Where called for in the Contract, conical steel pile tips shall be used when driving steel casings. The tips shall be inside fit, flush-mounted such that the tip and/or weld bead does not protrude more than 1⁄16 inch beyond the nominal outside diameter of the steel casing.

If conical tips are not specified, the lower end of each casing shall have a steel driving plate that is thick enough to keep the casing watertight and free from distortion as it is driven. The diameter of the steel driving plate shall not be greater than the outside diameter of the steel casing.

Where called for in the Contract, inside-fit cutting shoes shall be used when driving open-ended steel piles. The cutting shoes shall be flush-mounted such that the shoe and/or weld bead does not protrude more than 1⁄16 inch beyond the nominal outside diameter of the steel pile. The cutting shoe shall be of an inside diameter at least ¾ inch less than the nominal inside diameter of the steel pile.

Pile tips or shoes shall be of a type denoted in the Qualified Products List. If pile tips or shoes other than those denoted in the Qualified Products List are proposed, the Contractor shall submit Type 2 Working Drawings consisting of shop drawings of the proposed pile tip along with design calculations, Specifications, material chemistry and installation requirements, along with evidence of a pile driving test demonstrating suitability of the proposed pile tip. The test shall be performed in the presence of the Engineer or an acceptable independent testing agency. The test shall consist of driving a pile fitted with the proposed tip. If the pile cannot be visually inspected (Section 6-05.3(11)F), a sacrificial pile fitted with the proposed tip shall be driven outside the proposed foundation limits. The pile shall be driven to a depth sufficient to develop the required nominal bearing resistance as called for in the Contract, in ground conditions determined to be equivalent to the ground conditions at the project site. For closed-ended casings or piles, the pile need not be removed if, in the opinion of the Engineer, the pile can be inspected for evidence of damage to the pile or the tip. For open-ended steel casings or piles, timber piles or H-piles, the pile shall be removed for inspection.

6-05.3(9) Pile Driving Equipment

6-05.3(9)A Pile Driving Equipment Approval

Prior to driving piles, the Contractor shall submit Type 2 Working Drawings consisting of details of each proposed pile driving system. The pile driving system shall meet the minimum requirements for the various combinations of hammer type and pile type specified in this section. These requirements are minimums and may need to be increased in order to ensure that the required nominal bearing resistance can be achieved, that minimum tip elevations can be reached, and to prevent pile damage.

The Contractor shall submit Type 2E Working Drawings consisting of a wave equation analysis for all pile driving systems used to drive piling with required maximum driving resistances of greater than 300 tons. The wave equation analysis shall be performed in accordance with the requirements of this section and the user's manual for the program. The wave equation analysis shall verify that the pile driving system proposed does not produce stresses greater than 50,000 psi or 90 percent of the yield stress whichever is less, for steel piles, or steel casings for cast-in-place concrete piles. For prestressed concrete piles, the allowable driving stress in kips shall be plus prestress in tension, and $0.85f'_c$ minus prestress in compression, where f'_c is the concrete compressive strength in kips per square inch. For precast concrete piles that are not prestressed, the allowable driving stress shall be 70 percent of the yield stress of the steel reinforcement in tension, and $0.85f'_c$ in compression. The wave equation shall also verify that the pile driving system does not exceed the refusal criteria at the depth of penetration anticipated for achieving the required nominal bearing resistance and minimum tip elevation. Furthermore, the wave equation analysis shall verify that at the maximum driving resistance specified in the Contract, the driving resistance is 100 blows per foot or less. Unless otherwise specified in the Contract, or directed by the Engineer, the following default values shall be used as input to the wave equation analysis program:

Output option (IOUT)	0
Factor of safety applied to (R_{ult})	1.0
Type of damping	Smith
Residual stress option	No

R_{ult} is the resistance of the pile used in the wave equation analyses. If the nominal bearing resistance equals the maximum driving resistance, a setup factor of 1.3 may be used in the wave equation analysis to account for pile setup. To use a setup factor in the wave equation analysis, R_{ult} in the analysis is the nominal bearing resistance divided by 1.3. If the maximum driving resistance exceeds the nominal bearing resistance, no setup factor should be used, and R_{ult} is equal to the maximum driving resistance of the pile.

Hammer Efficiencies	For Analysis of Driving Resistance	For Analysis of Driving Stresses
Single acting diesel hammers	0.72	0.84
Closed-ended diesel hammers	0.72	0.84
Single acting air/steam hammers	0.60	0.70
Double acting air/steam hammers	0.45	0.53
Hydraulic hammers or other external combustion hammers having ram velocity monitors that may be used to assign an equivalent stroke.	0.85	1.00

Changes to the pile driving system after completion of the Working Drawing review require a revised Working Drawing Submittal.

6-05.3(9)B Pile Driving Equipment Minimum Requirements

For each drop hammer used, the Contractor shall weigh it in the Engineer's presence or submit a Type 1 Working Drawing consisting of a certificate of its weight. The exact weight shall be stamped on the hammer. Drop hammers shall weigh not less than:

1. 3,000 pounds for piles under 50 feet long that have a nominal bearing resistance of not more than 60 tons, and
2. 4,000 pounds for piles 50 feet and longer or that have a nominal bearing resistance of 60 to 90 tons.

If a drop hammer is used for timber piles, it is preferable to use a heavy hammer and operate with a short drop.

For each diesel, hydraulic, steam, or air-driven hammer used, the Contractor shall submit a Type 1 Working Drawing consisting of the manufacturer's Specifications and catalog. These shall show all data needed to calculate the developed energy of the hammer used.

Underwater hammers may be used only with permission of the Engineer.

Drop hammers on timber piles shall have a maximum drop of 10 feet. Drop hammers shall not be used to drive timber piles that have nominal bearing resistance of more than 60 tons.

When used on timber piles, diesel, hydraulic, steam, or air-driven hammers shall provide at least 13,000 foot-pounds of developed energy per blow. The ram of diesel hammers shall weigh at least 2,700 pounds.

Precast concrete and precast-prestressed concrete piles shall be driven with a single-acting steam, air, hydraulic, or diesel hammer with a ram weight of at least half as much as the weight of the pile, but never less than the minimums stated below. The ratio of developed hammer energy to ram weight shall not exceed 6. Steel casings for cast-in-place concrete, steel pipe, and steel H-piles shall also be driven with diesel, hydraulic, steam, or air hammers. These hammers shall provide at least the following developed energy per blow:

Minimum Developed Energy per Blow (ft-lbs)

Maximum Driving Resistance (Tons)	Air or Steam Hammers	Open Ended Diesel Hammers	Closed Ended Diesel Hammers	Hydraulic Hammers
Up to 165	21,500	23,000	30,000	18,500
166 to 210	27,500	29,500	38,000	23,500
211 to 300	39,000	41,500	54,000	33,500
301 to 450	59,000	63,000	81,000	50,500

In addition, the ram of diesel or hydraulic hammers shall have the following minimum weights:

Maximum Driving Resistance (Tons)	Minimum Ram Weight (lbs)
Up to 165	2,700
166 to 210	4,000
211 to 300	5,000
301 to 450	6,500

These requirements for minimum hammer size may be waived if a Type 2E Working Drawing is submitted consisting of a wave equation analysis demonstrating the ability of the hammer to obtain the required bearing resistance and minimum tip elevation without damage to the pile.

Vibratory hammers may be used to drive piles provided the location and plumbness requirements of this section are met. The required bearing resistance for all piles driven with vibratory hammers will be determined according to Section 6-05.3(12) by driving the pile at least an additional 2 feet using an impact hammer. This method of determining bearing resistance will be accepted provided the blows per inch are either constant or increasing. If the pile cannot be driven 2 feet, the pile will be considered acceptable for bearing if the pile is driven to refusal.

If water jets are used, the number of jets and water volume and pressure shall be enough to erode the material next to the pile at the tip. The equipment shall include a minimum of two water-jet pipes and two ¾ inch jet nozzles. The pump shall produce a constant pressure of at least 100 psi at each nozzle.

6-05.3(9)C Pile Driving Leads

All piles shall be driven with fixed-lead drivers. The leads shall be fixed on the top and bottom during the pile driving operation. Leads shall be long enough to eliminate the need for followers (except for timber piles as specified in Section 6-05.3(11)E). To avoid bruising or breaking the surface of treated timber piles, the Contractor shall use spuds and chocks as little as possible. In building a trestle or foundation with inclined piles, leads shall be adapted for driving batter piles.

A helmet of the right size for the hammer shall distribute the blow and protect the top of steel piling or casings from driving damage. The helmet shall be positioned symmetrically below the hammer's striking parts, so that the impact forces are applied concentric to the pile top.

Pile driving leads other than those fixed at the top and bottom may be used to complete driving, if permitted by the Engineer, when all of the following criteria are met:

1. Each plumb and battered pile is located and initially driven at least 20 feet in true alignment using fixed leads or other approved means.
2. The pile driving system (hammer, cushion and pile) will be analyzed by Pile Driving Analyzer (PDA) to verify driving stresses in the pile are not increased due to eccentric loading during driving, and transferred hammer energy is not reduced due to eccentric loading during driving, for all test piles and at least one production pile per pier. Unless otherwise specified, the cost of PDA testing shall be incidental to the various unit Contract prices for driving piles.

6-05.3(10) Test Piles

If the Contract or the Engineer call for it, the Contractor shall drive test piles to determine pile lengths required to reach the required nominal bearing resistance, penetration, or both. Test piles shall be:

1. Made of the same material and have the same tip diameter as the permanent piles (although test piles for treated timber piles may be either treated or untreated);
2. Driven with pile tips if the permanent piles will have tips;
3. Prebored when preboring is specified for the permanent piles;
4. Identical in cross-section and other characteristics to the permanent piles when the test piles are steel casings for cast-in-place concrete piles, precast concrete, precast-prestressed concrete or steel pipe or H-pile;
5. Long enough to accommodate all soil conditions;
6. Driven with equipment and methods identical to those to be used for the permanent piles;
7. Located as the Engineer directs; and
8. Driven before permanent piles in a given pier.

Test piles may also be driven by the Contractor (at no cost to the Contracting Agency) as evidence that the pile driving system selected will not damage the pile or result in refusal prior to reaching the specified minimum tip elevation.

Timber test piles shall be driven outside the footing and cut off 1 foot below the finished ground line. Timber test piles shall not be used in place of permanent piles.

Steel and all types of concrete test piles shall become permanent piles. The Contracting Agency has reduced the number of permanent piles by the number of test piles.

The Contractor shall base test pile length on test-hole data in the Contract. Test piles that prove to be too short shall be replaced (or spliced if the Contract allows splicing) at the Contractor's expense.

In foundations and trestles, test piles shall be driven to at least 15 percent more than the nominal bearing resistance required for the permanent piles, except where pile driving criteria is determined by the wave equation. When pile driving criteria is specified to be determined by the wave equation, the test piles shall be driven to the same nominal bearing resistance as the production piles. Test piles shall penetrate to a minimum tip elevation specified in the Contract. If no minimum tip elevation is specified, test piles shall extend at least 10 feet below the bottom of the concrete footing or ground line, and 15 feet below the bottom of the concrete seal.

When a test pile to be left as a permanent pile has been so damaged by handling or driving that the Engineer believes it unfit for use, the Contractor shall remove and replace the pile at no additional cost to the Contracting Agency. The Engineer may direct the Contractor to overdrive the test pile to more than 15 percent above the nominal bearing resistance for permanent piles, or if the wave equation is used to determine driving criteria, the Engineer may direct the Contractor to overdrive the test pile above the nominal bearing resistance. In these cases, the overdriving shall be at the Contractor's expense. But if pile damage results from this overdriving, removal and replacement will be at the Contracting Agency's expense.

6-05.3(11) Driving Piles

6-05.3(11)A Tolerances

For elevated pier caps, the tops of piles at cut-off elevation shall be within 2 inches of the horizontal locations indicated in the Contract. For piles capped below final grade, the tops of piles at cut-off elevation shall be within 6 inches of the horizontal locations indicated in the Contract. No pile edge shall be nearer than 4 inches from the edge of a footing or cap. Piles shall be installed such that the axial alignment of the top 10 feet of the pile is within 4 percent of the specified alignment. No misaligned steel or concrete piles shall be pulled laterally. A properly aligned section shall not be spliced onto a misaligned section for any type of pile. Unless the Contract shows otherwise, all piles shall be driven vertically.

6-05.3(11)B Foundation Pit Preparation

The Contractor shall replace (and bear the cost of replacing) piles damaged or destroyed before or during driving.

The Contractor shall completely dig all foundation pits (and build any required cofferdams or cribs) before driving foundation piles. The Contractor shall adjust pit depths to allow for upheaval caused by pile-driving, judging the amount of adjustment by the nature of the soil. Before constructing the footing or pile cap, the Contractor shall restore the pit bottom to correct elevation by removing material or by backfilling with granular material.

6-05.3(11)C Preparation for Driving

Treated and untreated timber piles shall be freshly cut square on the butt ends just before they are driven. If piles will be driven into hard material, caps, collars, or bands shall be placed on the butt ends to prevent crushing or brooming. If the head area of the pile is larger than that of the hammer face, the head shall be snipped or chamfered to fit the hammer. On treated piles, the heads shall be snipped or chamfered to at least the depth of the sapwood to avoid splitting the sapwood from the pile body.

The Contractor shall match timber pile sizes in any single bent to prevent sway braces from undue bending or distorting.

When driven, pile faces shall be turned as shown in the Plans or as the Engineer directs.

No precast-prestressed pile shall be driven until test cylinders poured with it reach at least the specified compressive strength shown in the Contract. On all other precast piles, the cylinders must reach a compressive strength of at least 4,000 psi before the piles are driven.

Helmets of approved design shall protect the heads of all precast concrete piles as they are driven. Each helmet shall have fitted into it a cushion next to the pile head. The bottom side of the helmet shall be recessed sufficiently to accommodate the required pile cushion and hold the pile in place during positioning and driving. The inside helmet diameter shall be determined before casting the pile, and the head of the pile shall be formed to fit loosely inside the helmet.

Steel Casing, steel pipe or H-piles shall have square-cut ends.

6-05.3(11)D Achieving Minimum Tip Elevation and Bearing

Once pile driving has started, each pile shall be driven continuously until the required nominal bearing resistance shown in the Contract has been achieved. Pauses during pile driving, except for splicing, mechanical breakdown, or other unforeseen events, shall not be allowed.

If the Contract specifies a minimum tip elevation, the pile shall be driven to at least the minimum tip elevation, even if the nominal bearing resistance has been achieved, unless the Engineer directs otherwise. If a pile does not develop the required nominal bearing resistance at the minimum tip elevation, the Contractor shall continue driving the pile until the required bearing resistance is achieved. If no minimum tip elevation is specified, then the piles shall be driven to the nominal bearing resistance shown in the Contract and the following minimum penetrations:

Pile supporting cross-beams, bents, elevated pile caps elevation	10 feet below final top of ground
Piles supporting foundations	10 feet below bottom of foundation
Piles with a concrete seal	15 feet below bottom of seal

If overdriving is required in order to reach a specified minimum tip elevation, the Contractor shall provide a pile driving system which will not result in damage to the pile or refusal before the minimum tip elevation is reached. The cost of overdriving shall be incidental to the various unit Contract prices for furnishing and driving piles.

So long as the pile is not damaged and the embankment or foundation material being driven through is not permanently damaged, the Contractor shall use normal means necessary to:

1. Secure the minimum depth specified,
2. Penetrate hard material that lies under a soft upper layer,
3. Penetrate through hard material to obtain the specified minimum tip elevation, or
4. Penetrate through a previously placed embankment.

Normal means refer to methods such as preboring, spudding, or jetting piles. Blasting or drilling through obstructions are not considered normal means.

Prebored holes and pile spuds shall have a diameter no larger than the least outside dimension of the pile. After the pile is driven, the Contractor shall fill all open spaces between the pile and the soil caused by the preboring or spudding with dry sand, or pea gravel, or controlled density fill as approved by the Engineer.

If water jets are used, the jets shall be withdrawn before the pile reaches its final penetration, and the pile shall then be driven to its final penetration and nominal bearing resistance. The pile shall be driven a minimum of 2 feet to obtain the nominal bearing resistance after the jets are withdrawn, or to refusal, whichever occurs first. If the water jets loosen a pile previously driven, it shall be redriven in place or pulled and replaced by a new pile. To check on pile loosening, the Contractor shall attempt to redrive at least one in every five piles, but no less than one pile per bent or pier.

The various unit Contract prices for driving piles shall cover all costs related to the use of water jets, preboring, or spudding. The Contracting Agency will not pay for costs the Contractor incurs in redriving piles loosened as a result of using water jets, preboring, or spudding.

If the Engineer requires, the Contractor shall overdrive the pile beyond the nominal bearing resistance and minimum tip elevation shown in the Contract. In this case, the Contractor will not be required to:

1. Use other than normal means to achieve the additional penetration,
2. Bear the expense of removing or replacing piles damaged by overdriving, or
3. Bear the expense of overdriving the pile more than 3 feet as specified in Section 6-05.5.

In driving piles for footings with seals, the Contractor shall use no method (such as jetting or preboring) that might reduce friction resistance.

6-05.3(11)E Use of Followers for Driving

Followers shall not be used to drive concrete or steel piles. On timber piles, the Contractor may use steel (not wooden) followers if the follower fits snugly over the pile head. If a follower is used, the Contractor shall, in every group of 10 piles, drive one long pile without a follower, but no less than one pile per bent or pier, to the required nominal bearing resistance and minimum tip elevation. This long pile shall be used to test the bearing resistance of the piles driven with a follower in the group. The tip elevation of the long pile shall be similar to the elevation of the piles driven with the follower. If the tip elevations are significantly different, as determined by the Engineer, the Contractor shall redrive the remaining piles in the group to the tip elevation of the longer pile.

6-05.3(11)F Pile Damage

The Contractor shall remove and replace (and bear the cost of doing so) piles that are damaged as determined by the Engineer.

After driving a steel casing for a cast-in-place concrete pile, the Contractor shall leave it empty until the Engineer has inspected and accepted it. The Contractor shall make available to the Engineer a light suitable for inspecting the entire length of its interior. The Engineer will reject casings that are improperly driven, that show partial collapse that would reduce its nominal bearing resistance, or that has been reduced in diameter, or that will not keep out water. The Contractor shall replace (and bear the cost of replacing) all rejected casings.

Pile heads which have been broomed, rolled, or otherwise significantly damaged as determined by the Engineer shall be cut back to undamaged material before proceeding with driving as well as final acceptance of the pile.

6-05.3(11)G Pile Cutoff

The Contractor shall trim the tops of all piles to the true plane shown in the Contract and to the elevation the Engineer requires. If a pile is driven below cutoff elevation without the Engineer's permission, the Contractor shall remove and replace it (and bear the costs of doing so), even if this requires a longer pile. Piles that rise as nearby piles are driven, shall be driven down again if the Engineer requires.

Piles under timber caps or grillages shall be sawed to the exact plane of the Structure above them and fit it exactly. No shimming on top of timber piles to adjust for inaccurate pile top elevations will be permitted. If a timber pile is driven out of line, it shall be straightened without damage before it is cut off or braced.

Steel casing shall be cut off at least 6 inches below the finished ground line or at the low water line if the casing will be visible as determined by the Engineer.

6-05.3(11)H Pile Driving From or Near Adjacent Structures

The Contractor shall not drive piling from an existing Structure unless all of the following conditions are met:

1. The existing Structure will be demolished within the Contract;
2. The existing Structure is permanently closed to traffic; and
3. Type 2E Working Drawings are submitted in accordance with Sections 1-05.3 and 6-02.3(16), showing the structural adequacy of the existing Structure to safely support all of the construction loads.

Freshly placed concrete in the vicinity of the pile driving operation shall be protected against vibration in accordance with Section 6-02.3(6)D.

6-05.3(12) Determination of Bearing Values

The following formula shall be used to determine nominal bearing resistances:

$$P = F \times E \times Ln(10N)$$

Where:

P	=	nominal bearing resistance, in tons
F	=	1.8 for air/steam hammers
	=	1.2 for open ended diesel hammers and precast concrete or timber piles
	=	1.6 for open ended diesel hammers and steel piles
	=	1.2 for closed ended diesel hammers
	=	1.9 for hydraulic hammers
	=	0.9 for drop hammers
E	=	developed energy, equal to W times H[1], in ft-kips
W	=	weight of ram, in kips
H	=	vertical drop of hammer or stroke of ram, in feet
N	=	average penetration resistance in blows per inch for the last 4 inches of driving
Ln	=	the natural logarithm, in base "e"

[1]For closed-end diesel hammers (double-acting), the developed hammer energy (E) is to be determined from the bounce chamber reading. Hammer manufacturer calibration data may be used to correlate bounce chamber pressure to developed hammer energy. For double acting hammer hydraulic and air/steam hammers, the developed hammer energy shall be calculated from ram impact velocity measurements or other means acceptable to the Engineer. For open ended diesel hammers (single-acting) use the blows per minute to determine the developed energy (E).

The above formula applies only when:

1. The hammer is in good condition and operating in a satisfactory manner.
2. A follower is not used.
3. The pile top is not damaged.
4. The pile head is free from broomed or crushed wood fiber.
5. The penetration occurs at a reasonably quick, uniform rate; and the pile has been driven at least 2 feet after an interruption in driving greater than 1 hour in length.

6. There is no perceptible bounce after the blow. If a significant bounce cannot be avoided, twice the height of the bounce shall be deducted from "H" to determine its true value in the formula.
7. For timber piles, bearing resistances calculated by the formula above shall be considered effective only when it is less than the crushing strength of the piles.
8. If "N" is greater than or equal to 1.0 blow/inch.

If "N" required to achieve the required nominal bearing resistance using the above formula is less than 1.0 blow/inch, the pile shall be driven until the penetration resistance is a minimum of 1.0 blow/inch for the last 2 feet of driving.

The Engineer may require the Contractor to install a pressure gauge on the inboard end of the hose to check pressure at the hammer.

If water jets are used in driving, bearing resistances shall be determined either: (1) by calculating it with the driving data and the formula above after the jets have been withdrawn and the pile is driven at least 2 feet, or (2) by applying a test load.

6-05.3(13) Treatment of Timber Pile Heads

After cutting timber piles to correct elevation, the Contractor shall thoroughly coat the heads of all untreated piles with two coats of an approved preservative that meets the requirements of Section 9-09 (except concrete-encased piles).

After cutting treated timber piles to correct elevation, the Contractor shall brush three coats of a preservative that meets the requirements of Section 9-09 on all pile heads (except those to be covered with concrete footings or concrete caps). The pile heads shall then be capped with alternate layers of an approved roofing asphalt and a waterproofing fabric that conforms to Section 9-11.1. The cap shall be made of four layers of an approved roofing asphalt and three layers of fabric. The fabric shall be cut large enough to cover the pile top and fold down at least 6 inches along all sides of the pile. After the fabric cover is bent down over the pile, its edges shall be fastened with large-head galvanized nails or with three turns of galvanized wire. The edges of the cover shall be neatly trimmed.

On a treated timber pile encased in concrete, the cut end shall receive two coats of an approved preservative that meets the requirements of Section 9-09 and then a heavy coat of an approved roofing asphalt.

6-05.3(14) Extensions and Buildups of Precast Concrete Piles

The Contractor shall add extensions, or buildups (if necessary) on precast concrete piles after they are driven to the required nominal bearing resistance and minimum tip elevation.

Before adding extensions or buildups to precast-prestressed piles, the Contractor shall remove all spalled concrete, leaving the pile fresh-headed and with a top surface perpendicular to the axis of the pile. The concrete in the buildup shall be Class 5000.

Before adding to non-prestressed precast concrete piles, the Contractor shall cut the pile head away to a depth 40 times the diameter of the vertical reinforcing bar. The final cut shall be perpendicular to the axis of the pile. Reinforcement of the same density and configuration as used in the pile shall be used in the buildup and shall be fastened firmly to the projecting steel. Forms shall be placed to prevent concrete from leaking along the pile. The concrete in the buildup shall be Class 4000.

Just before placing the concrete for extensions or buildups to precast or precast-prestressed concrete piles, the Contractor shall thoroughly wet the top of the pile. Forms shall remain in place at least 3 days.

6-05.3(15) Completion of Cast-In-Place Concrete Piles

After acceptance by the Engineer, driven casings shall be cut off horizontally at the required elevation. They shall be clean and free of water when concrete and reinforcing steel are placed.

These piles shall consist of steel casings driven into the ground, reinforced as specified, and filled with Class 5000P concrete.

6-05.3(15)A Reinforcement

All bars shall be fastened rigidly into a single unit, then lowered into the casing before the concrete is placed. Loose bars shall not be used.

Spiral hooping reinforcement shall be deformed steel bar, plain steel bar, cold-drawn wire, or deformed wire.

6-05.3(15)B Placing Concrete

Before placing concrete, the Contractor shall remove all debris and water from the casing. If the water cannot be removed, the casing shall be removed (or cut off 2 feet below the ground and filled with sand) and a new one driven.

The Contractor shall place concrete continuously through a 5-foot rigid conduit directing the concrete down the center of the pile casing, ensuring that every part of the pile is filled and the concrete is worked around the reinforcement. The top 5 feet of concrete shall be placed with the tip of the conduit below the top of fresh concrete. The Contractor shall vibrate, as a minimum, the top 10 feet of concrete. In all cases, the concrete shall be vibrated to a point at least 5 feet below the original ground line.

6-05.4 Measurement

Measurement for driving (type) pile will be the number of piles driven in place.

In these categories, measurement will be the longer of either the number of linear feet driven below cutoff or as shown in the Engineer's order list:

1. Furnishing timber piling (untreated or name of treatment).
2. Precast concrete and precast-prestressed concrete piling.

In these categories, measurement will be the number of linear feet driven below cutoff, but no Engineer's order list will be provided:

1. Cast-in-place concrete piling.
2. Furnishing steel piling.

Measurement for furnishing and driving test piles will be the number actually furnished and driven as the Contract requires.

Measurement for steel pile tips or shoes will be by the number of tips or shoes actually installed and driven in place on steel casings or steel piles.

6-05.5 Payment

Payment will be made for each of the following Bid items that are included in the Proposal:

"Furnishing and Driving (type) Test Pile", per each.

The unit Contract price per each for "Furnishing and Driving (type) Test Pile" shall be full pay for furnishing and driving test piles to the nominal bearing resistance or penetration required by the Engineer, furnishing and installing a pile tip when pile tips are specified for the permanent piles, preboring when preboring is specified for the permanent piles, for pulling the piles or cutting them off as required, and for removing them from the site or for delivery to the Contracting Agency for salvage when ordered by the Engineer. For cast-in-place concrete test piles, this price shall include furnishing, fabricating, and installing the steel reinforcing bar cage, and furnishing, casting, and curing the concrete. This price shall also include all costs in connection with moving all pile driving equipment or other necessary equipment to the site of the Work and for removing all such equipment from the site after the piles have been driven. If, after the test piles have been driven, it is found necessary to eliminate the piling from all or any part of the Structure, no additional pay will be allowed for moving the pile driving equipment to and from the site of the Work.

“Driving Timber Pile (untreated or name treatment)”, per each.

The unit Contract price per each for “Driving Timber (type) Pile” shall include all metal shoes which the Contractor has determined to be beneficial to the pile driving.

“Driving Conc. Pile (size)”, per each.

“Driving St. Pile”, per each.

The unit Contract price per each for “Driving (type) Pile (____)” shall be full pay for driving the pile to the nominal bearing and/or penetration specified.

“Furnishing Timber Piling (untreated or name treatment)”, per linear foot.

“Furnishing Conc. Piling (size)”, per linear foot.

“Furnishing St. Piling”, per linear foot.

The unit Contract price per linear foot for “Furnishing (type) Piling (____)” shall be full pay for furnishing the piling specified, including furnishing, fabricating, and installing the steel reinforcing bar cage, and furnishing casting, and curing the concrete, as required for concrete piling. Such price shall also be full pay, for furnishing timber, precast concrete, or precast-prestressed concrete piling length ordered from an Engineer’s order sheet but not driven.

“Precast Concrete Pile Buildup”, by force account.

Payment for buildups of precast or precast-prestressed concrete piles will be made on the basis of force account Work as covered in Section 1-09.6. No payment will be made for buildups or additional lengths of buildup made necessary because of damage to the piling during driving. The length of splice for precast concrete piles includes the length cut off to expose reinforcing steel for the splice. The length of splice for precast-prestressed piles includes the length in which holes are drilled and reinforcing bars are grouted.

For the purpose of providing a common Proposal for all Bidders, the Contracting Agency entered an amount for “Precast Concrete Pile Buildup” in the Proposal to become part of the total Bid by the Contractor.

“Furnishing Steel Pile Tip or Shoe (size)”, per each.

6-06 Bridge Railings

6-06.1 Description

This Work consists of providing and building bridge railings that meet the requirements of the Plans, these Specifications, and the Engineer.

6-06.2 Materials

Materials shall meet the requirements of the following sections:

Timber Railing	9-09
Metal Railing	9-06.18

6-06.3 Construction Requirements

6-06.3(1) Timber Railings

Wheel guards and railings shall be true to line and grade and framed accurately. The Contractor shall follow Section 6-04 whenever this Subsection does not specify a construction method.

Unless the Plans show otherwise, wheel guards shall be:

1. Beveled and surfaced on the Roadway side and surfaced on the top edge. They may be surfaced on four sides (S4S).
2. Laid in sections at least 12 feet long.
3. Bolted through the floor plank and outside stringer (or nailing piece) with ¾ inch diameter bolts spaced no more than 4 feet apart.

All rails and rail post material shall be S4S and painted as required in Section 6-07. Railing members shall be fastened securely together, with the bolts tightened once at installation and again just before the Contracting Agency's final acceptance of the Contract.

6-06.3(2) Metal Railings

Metal railing includes posts, web members, and horizontal members of the sidewalk and Roadway railing. Unless the Plans or Special Provisions show otherwise, these shall be made of aluminum alloy or steel.

Before fabricating the railing, the Contractor shall submit Type 2 Working Drawings of the shop plans. The Contractor may substitute other rail connection details for those shown in the Plans if details of these changes show in the shop plans and if the Engineer accepts them in the Working Drawing response comments. In reviewing the shop plans, the Engineer indicates only that they are adequate and complete enough. The review does not indicate a check on dimensions.

Anchor bolts shall be positioned with a template to ensure that bolts match the hole spacing of the bottom channels or anchorage plates.

Where specified, cover plates shall fit the bottom channel tightly after being snapped into position.

Metal railings shall be installed true to line and grade (or camber). After first setting the railing, the Contractor shall readjust all or part of it, if necessary, to create an overall line and grade pleasing to the eye.

6-06.4 Measurement

Timber railing will be measured by the thousand board feet (MBM) as shown in Section 6-04.

Metal railing will be measured by the linear foot along the line and slope at the base of the completed railing.

6-06.5 Payment

Payment will be made for each of the following Bid items that are included in the Proposal:

"Timber and Lumber (untreated or name treatment)", per MBM.

"Bridge Railing Type ____", per linear foot.

In case no item is included in the Contract for "Bridge Railing Type ____" and payment is not otherwise provided, all metal railings shall be included in the lump sum Contract price for "Structural Carbon St". as specified in Section 6-03.

6-07 Painting

6-07.1 Description

This work consists of containment, surface preparation, shielding adjacent areas from work, testing and disposing of debris, furnishing and applying paint, and cleaning up after painting is completed. The work shall comply with all requirements of the Plans, these Specifications, and the Engineer. Terminology used herein is in accordance with the definitions used in Volume 2, Systems and Specifications, of the SSPC *Steel Structures Painting Manual*.

6-07.2 Materials

Materials shall meet the requirements of the following sections:

Paints and Related Materials	9-08
Powder Coating Materials for Coating Galvanized Surfaces	9-08.2
Abrasive Blast Media	9-08.4(1)
Lead Abatement Additive	9-08.4(2)
Bird Guano Treatment	9-08.5(1)
Fungicide Treatment	9-08.5(2)
Water	9-08.5(3)
Filter Fabric	9-08.6
Single Component Polyurethane Sealant	9-08.7
Foam Backer Rod	9-08.8

6-07.3 Construction Requirements

6-07.3(1) Work Force Qualifications

6-07.3(1)A Work Force Qualifications for Shop Application of Paint

Facilities for shop application of paint shall either be selected from one of the facilities listed in the WSDOT Qualified Products List as an approved coating facility for new steel structures or shall be approved through the WSDOT Request for Approval of Material process. The work force may be accepted based on the approved facility.

6-07.3(1)B Work Force Qualifications for Field Application of Paint

The Society for Protective Coatings (SSPC) and the National Association of Corrosion Engineers (NACE) merged to form the Association for Materials Protection and Performance (AMPP). Documents submitted under SSPC, NACE, or AMPP are acceptable.

The Contractor preparing the surface and applying the paint shall be certified under AMPP (formerly SSPC)-QP 1 or NACE International Institute Contractor Accreditation Program (NIICAP) AS 1.

The Contractor removing and otherwise disturbing existing paint containing lead and other hazardous materials shall be certified under AMPP (formerly SSPC)-QP 2, Category A or NIICAP AS 2.

6-07.3(2) Submittals

The Contractor shall submit a painting plan consisting of one comprehensive submittal including all components described in this Section. The Contractor shall submit Type 2 Working Drawings of the painting plan components, except containment system and support and platform plans as listed in Section 6-07.3(2)F item number 3 shall be Type 2E Working Drawings. Each component of the plan shall identify the specification section it represents and shall be assembled in an order consistent with submittal requirement sections.

For shop application of paint, the painting plan shall include the documents and samples listed in Sections 6-07.3(2)B, 6-07.3(2)C, and 6-07.3(2)E.

For field application of paint, the painting plan shall include the documents and samples listed in Section 6-07.3(2)A through 6-07.3(2)F.

6-07.3(2)A Work Force Qualifications Submittal Component

The work force qualifications submittal component of the painting plan shall include the following:

1. Documentation of the Contractor's workforce qualifications as specified in Section 6-07.3(1).
2. Resumé of qualifications and contact information for the Contractor's on-site supervisors. Each on-site supervisor shall have 3 years' minimum of industrial painting field experience with 1 year minimum of field supervisory or management experience in bridge painting projects.

6-07.3(2)B Contractor's Quality Control Program Submittal Component

The Contractor's quality control program submittal component of the painting plan shall include the following for each phase of work:

1. Frequency of quality control inspection.
2. Description of the inspection process and techniques.
3. List of inspection tools and equipment.
4. Acceptance criteria for each item of work.
5. Procedure for implementation of corrective action for non-conforming work.
5. Example of each completed form(s) of the daily quality control report used to document the inspection work and tests performed by the Contractor's quality control personnel.

6-07.3(2)C Paint System Manufacturer and Paint System Information Submittal Component

The paint system manufacturer and paint system information submittal component of the painting plan shall include the following:

1. Product data sheets and Safety Data Sheets (SDS) on the paint materials, paint preparation, and paint application, as specified by the paint manufacturer, including:
    a. All application instructions, including the mixing and thinning directions.
    b. Recommended spray nozzles and pressures.
    c. Minimum and maximum drying time between coats.
    d. Restrictions on temperature and humidity.
    e. Repair procedures for shop and field applied coatings.
    f. Maximum dry film thickness for each coat.
    g. Minimum wet film thickness for each coat to achieve the specified minimum dry film thickness.
2. Identification of, and contact information for, the paint system manufacturer's technical representative.
3. For painting of new steel, the friction coefficient of the faying surface, including test results and the paint manufacturer's Certificate of Compliance in support of the friction coefficient. Certificate of Compliance shall be completed within the last 84 months and be in accordance with the Specifications for Structural Joints Using High-Strength Bolts, 2020 as approved by the Research Council on Structural Connections.

6-07.3(2)D Hazardous Waste Containment, Collection, Testing, and Disposal Submittal Component

The hazardous waste containment, collection, testing, and disposal shall meet all Federal and State requirements, and the submittal component of the painting plan shall include the following:

1. Abrasive blasting containment system attachment and support in accordance with Section 6-07.3(10)A, with a complete description of each attachment device.
2. Details of jobsite material storage facilities and containment waste storage facilities, including location, security, and environmental control.
3. Methods and materials used to contain, collect, and dispose of all containment waste and all construction-related waste, including transportation of waste.
4. Details of the containment waste sampling plan conforming to WAC 173-303 for waste designated as dangerous waste or extremely hazardous waste.
5. The name of, and contact information for, the accredited analytical laboratory performing the testing of the containment waste samples in accordance with Section 6-07.3(10)F.
6. Process for tracking the disposal of hazardous waste, including a sample form of the tracking documentation.
7. When a wind speed threshold is specified, a description of the method to lower or withdraw tarps, plastic exterior, and other containment components presenting an exposed face to wind, and the estimated time required to accomplish this action.
8. Provisions for dust and debris collection, ventilation, and auxiliary lighting within the containment system. The plan shall include a minimum calculation of airflow in accordance with section 10.6 of SSPC Guide 16. If the containment is designed to be erected in sections, the airflow shall be calculated for each section. The Contractor shall provide reliable wait times for the ventilation system to reduce lead concentrations below the permissible exposure limit and action limit established in WAC 296-155-176 and applicable exposure limits in WAC 296-841. The Contractor shall assume inspectors will be in the containment for up to two hours. The wait times will be developed by either a licensed mechanical engineer or a certified industrial hygienist accredited by the Board for Global EHS Credentialing.

6-07.3(2)E Cleaning and Surface Preparation Submittal Component

The cleaning and surface preparation submittal component of the painting plan shall include the following:

1. Details of the abrasive blast cleaning operation, including:
    a. Description of the abrasive blast cleaning procedure.
    b. Type, manufacturer, and brand of abrasive blast material and all associated additives, including Safety Data Sheets (SDS).
    c. Description of the abrasive blast cleaning equipment to be used.

6-07.3(2)F Paint Application Equipment and Operations Submittal Component

The paint application equipment and operations submittal component of the painting plan shall include the following:

1. Description of the equipment used for paint application operations.
2. Details of jobsite material storage facilities, including location, security, and environmental control.

3. A Type 2E Working Drawing with a description of the supports and platforms used to support equipment, materials, and workers, including scaffolds, platforms, accordion lifts, and barges, and the methods used to attach, moor, and anchor these supports and platforms.
4. Drip tarps in accordance with Section 6-07.3(10)O.
5. Methods and materials used to protect surrounding structures, equipment, and property from exposure to, and damage from, painting operations.
6. Details of paint application operations for areas of limited and restricted access.
7. Description of the method for the removal of any accidental spills or drips on traffic that occur during the normal painting operations, and provisions for providing a vehicle-cleaning station.

6-07.3(2)G Painting Plan Meeting

At the option of the Contracting Agency, a painting plan meeting may be scheduled following review of the Contractor's initial submittal of the plan. The Contractor shall be represented by the superintendent, on-site supervisors, and quality control inspectors.

6-07.3(3) Quality Control and Quality Assurance

6-07.3(3)A Quality Control and Quality Assurance for Shop Application of Paint

For shop application of paint, quality control procedures shall be as accepted by the Engineer.

6-07.3(3)B Quality Control and Quality Assurance for Field Application of Paint

For field application of paint, the Contractor shall conduct quality control inspections as required by SSPC-PA 1, using the personnel and the processes outlined in the painting plan. The Contractor shall maintain current copies of the SSPC *Painting Manual*, Volumes 1 and 2, at the project site at all times. The Contractor's quality control operations shall include at a minimum monitoring and documenting the following for each working day:

1. Equipment, personnel, and materials used.
2. Environmental conditions (ambient air temperature and humidity, steel surface temperature, dew point, wind direction, and velocity).
3. Steel surface condition, profile, and preparation.
4. Paint application and film thickness.

A Type 1 Working Drawing consisting of the Contractor's daily quality control report, signed and dated by the Contractor's quality control inspector, accompanied by copies of the test results of quality control tests performed on the work covered by the daily quality control report, shall be submitted to the Engineer before the end of the next day's work shift.

The Contractor shall provide the Engineer time and access to perform quality assurance testing. Each painting operation phase shall be considered a hold point, from which the Contractor shall not proceed with continuing work until receiving the Engineer's acceptance.

The Engineer may perform quality assurance testing at each of the following phases of painting operations:

1. After SSPC-SP 1 cleaning.
2. After abrasive blast cleaning, hand and power tool surface cleaning, and compressed air surface cleaning.
3. After applying each coat when dry.
4. During final inspection of all work at the end of the project.

Quality assurance testing may include the following tests:

1. Environmental humidity conditions for painting in accordance with ASTM E337.
2. Cleanness of abrasive blasting media and ionic contamination of abrasive blasting media in accordance with ASTM D4940.
3. Cleanness of compressed air in accordance with ASTM D4285.
4. Pictorial of surface preparation guides in accordance with SSPC-VIS 1, 3, 4, and 5.
5. Surface profile by Keanne-Tator comparator in accordance with ASTM D4417 and SSPC PA17.
6. Surface profile by replica tape in accordance with ASTM D4417.
7. Wet film thickness in accordance with ASTM D4414.
8. Dry film thickness by magnetic gage in accordance with SSPC-PA 2 modified.
9. Dry film thickness by Tooke gage in accordance with ASTM D4138.

The Contractor shall repair all damage to paint resulting from Contracting Agency's quality assurance inspections at no additional cost or time to the Contracting Agency.

6-07.3(3)C Quality Control and Quality Assurance During Hold Point Inspection

The Contractor shall notify the Engineer once cleaning has been completed so that a hold point inspection may be performed. The Contractor shall conduct operations to ensure the concentration of contaminants is reduced below the action and permissible exposure limits for lead or other regulated air contaminants before requesting the Engineer to initiate a hold point inspection. Protocols may be modified by the Contracting Agency based on staff air monitoring to limit exposures from all airborne contaminants. The Contractor shall ensure the ventilation system functions at capacity to the established protocols. The Contractor shall ensure the ventilation remains in full operation at all times. The Contractor shall not conduct activities that generate regulated airborne contaminants while Contracting Agency personnel are in the containment.

6-07.3(4) Paint System Manufacturer's Technical Representative

The paint system manufacturer's technical representative shall be present at the jobsite for the pre-painting conference and for the first day of paint application, and shall be available to the Contractor and Contracting Agency for consultation for the full project duration.

6-07.3(5) Pre-Painting Conference

A pre-painting conference shall be held 5 to 10 working days before beginning painting operations to discuss the painting plan, construction operations, personnel, and equipment to be used. Those attending shall include:

1. (Representing the Contractor) The superintendent, on-site supervisors, and all crew members in charge of cleaning and preparing the surfaces, containing, collecting and disposing of all removed materials, applying the paint, and performing all quality control inspections, measurements and tests; and the paint system manufacturer's technical representative; and
2. (Representing the Contracting Agency) The Engineer, key inspection assistants, and representatives of the WSDOT HQ Construction Office.

If the Contractor's key personnel change throughout the performance of the Work, an additional conference shall be held if requested by the Engineer.

For projects that include painting of multiple structures, a separate conference may be held for each structure, at the discretion of the Engineer.

6-07.3(6) Paint Containers, Storage, and Handling

6-07.3(6)A Paint Containers

Paint container labels shall include the following information:

1. Manufacturer's name and product name, with batch number and date of manufacture.
2. Color name and SAE AMS Standard 595 color number, where applicable.
3. Shelf life of the product, from date of batch manufacture.
4. Storage requirements and temperature limits.

Paint containers shall conform to U.S. DOT hazardous material shipping regulations. Paint shall be delivered to the jobsite in the manufacturer's original unopened containers with the original manufacturer's label legible and intact. Paint will be rejected if the container has a puncture or if the lid shows signs of paint leakage. Each container shall be filled with paint and sealed airtight. Each container shall be filled with the amount of paint required to yield the specified quantity when measured at 70°F. All paint shall be shipped in new suitable containers having a capacity not greater than 5 gallons.

6-07.3(6)B Paint Storage

Paint materials shall not be used or stored on-site after the shelf life expiration date.

Paint material shipping, handling, and storage shall conform to Sections 1-06.4 and 9-08.1(4) and the following requirements:

1. Paint materials shall be stored in the manufacturer's original containers in a weather-tight space where the temperature is maintained within the storage temperature range recommended by the paint manufacturer, but in no case where the temperature is lower than 40°F or greater than 100°F.
2. The Contractor shall monitor and document daily the paint material storage facility with a high-low recording thermometer device.
3. The paint material storage facility shall be separate from the storage facilities used for storing painting equipment and used for storing containment waste and construction-generated waste.

6-07.3(7) Paint Sampling and Testing

The Contractor shall provide the Engineer 1 quart of each paint representing each lot. Samples shall be accompanied with a Safety Data Sheet.

If the quantity of paint required for each component of the paint system for the entire project is 20 gallons or less, then the paint system components will be accepted as specified in Section 9-08.1(7).

Sampling and testing performed by the Contracting Agency shall not be construed as determining or predicting the performance or compatibility of the individual paint or the completed paint system.

6-07.3(8) Equipment

6-07.3(8)A Paint Film Thickness Measurement Gages

Paint dry film thickness measurements shall be performed with either a Type 1 pull-off gage or a Type 2 electronic gage as specified in SSPC Paint Application Specification No. 2, Procedure for Determining Conformance to Dry Coating Thickness Requirements.

Paint wet film thickness measurement gages shall be stainless steel with notches graduated in 1-mil increments.

6-07.3(9) Painting New Steel Structures

All materials classified as nongalvanized structural steel shall be painted with a four-coat paint system as specified in Section 6-07.3(9)A. The primer coat shall be shop-applied. The intermediate, intermediate stripe, and top coats shall be field-applied after erection and following the primer coating repair operations, if any.

Steel surfaces embedded in concrete, and faying (contact) surfaces of bolted connections (including all surfaces internal to the connection and all filler plates) shall receive the primer coat only. Stainless steel surfaces are not required to be painted. Welded shear connectors are not required to be painted.

Temporary attachments or supports for scaffolding, containment or forms shall not damage the paint system.

6-07.3(9)A Paint System

The paint system applied to new steel surfaces shall consist of the following:

Option 1 (component based paint system):	
Primer Coat - Inorganic Zinc Rich	9-08.1(2)C
Intermediate Coat - Moisture Cured Polyurethane	9-08.1(2)G
Intermediate Stripe Coat - Moisture Cured Polyurethane	9-08.1(2)G
Top Coat - Moisture Cured Polyurethane	9-08.1(2)H
Option 2 (performance based paint system):	
Primer Coat - Inorganic Zinc Rich	9-08.1(2)M
Intermediate Coat - Epoxy	9-08.1(2)M
Intermediate Stripe Coat - Epoxy	9-08.1(2)M
Top Coat - Polyurethane	9-08.1(2)M

Paints and related materials shall be products listed in the current WSDOT Qualified Products List (QPL). Component based paint systems shall be listed on the QPL in the applicable sections of Section 9-08. Performance based systems shall be listed on the current Northeast Protective Coatings Committee (NEPCOAT) Qualified Products List “A” as listed on the WSDOT QPL in Section 9-08.1(2)M. If the paint and related materials for the component based system is not listed in the current WSDOT QPL, a sample shall be submitted to the State Materials Laboratory in Tumwater for evaluation and acceptance in accordance with Section 9-08.

All paint coating components of the selected paint system shall be produced by the same manufacturer. The paint system selected shall be used throughout the entire structure.

Paint formulations to be used on faying surfaces shall be Class B coatings with a mean slip coefficient not less than 0.50. The slip coefficient shall be determined by testing in accordance with “Appendix A - Test Method to Determine the Slip Coefficient for Coatings Used in Bolted Joints” included in the *Specification for Structural Joints Using High-Strength Bolts, June 2020* as approved by the Research Council on Structural Connections.

6-07.3(9)B Paint Color

Each successive coat shall be a contrasting color to the previously applied coat. The color of the top coat shall be as specified in the Plans or Special Provisions and shall conform to Section 9-08.1(8).

6-07.3(9)C Mixing and Thinning Paint

The Contractor shall thoroughly mix paint in accordance with the manufacturer’s written recommendations and by mechanical means to ensure a uniform and lump free composition. Paint shall not be mixed by means of air stream bubbling or boxing. Paint shall be mixed in the original containers and mixing shall continue until all pigment or metallic powder is in suspension. Care shall be taken to ensure that the solid material that has settled to the bottom of the container is thoroughly dispersed. After mixing, the

Contractor shall inspect the paint for uniformity and to ensure that no unmixed pigment or lumps are present.

Catalysts, curing agents, hardeners, initiators, or dry metallic powders that are packaged separately may be added to the base paint in accordance with the paint manufacturer's written recommendations and only after the paint is thoroughly mixed to achieve a uniform mixture with all particles wetted. The Contractor shall then add the proper volume of curing agent to the correct volume of base and mix thoroughly. The mixture shall be used within the pot life specified by the manufacturer. Unused portions shall be discarded at the end of each work day. Accelerants are not permitted except as allowed by the Engineer.

The Contractor shall not add additional thinner at the application site except as allowed by the Engineer. The amount and type of thinner, if allowed, shall conform to the manufacturer's specifications. If recommended by the manufacturer and allowed by the Engineer, a measuring cup with gradations in ounces shall be used for the addition of thinner to paint. No unmeasured addition of thinner to paint will be allowed. Paint found to be thinned by unacceptable methods will be rejected.

When recommended by the manufacturer, the Contractor shall constantly agitate paint during application by use of paint pots equipped with mechanical agitators.

The Contractor shall strain all paint after mixing to remove undesirable matter, but without removing the pigment or metallic powder.

Paint shall be stored and mixed in a secure, contained location to eliminate the potential for spills into State waters and onto the ground and highway surfaces.

6-07.3(9)D Coating Thickness

Dry film thickness shall be measured in accordance with SSPC Paint Application Specification No. 2, Procedure for Determining Conformance to Dry Coating Thickness Requirements.

The minimum dry film thickness of the primer coat shall not be less than 3.0 mils.

The minimum dry film thickness of each coat (combination of intermediate and intermediate stripe, and top) shall be not less than 3.0 mils.

The dry film thickness of each coat shall not be thicker than the paint manufacturer's recommended maximum thickness.

The minimum wet film thickness of each coat shall be specified by the paint manufacturer to achieve the minimum dry film thickness.

Film thickness, wet and dry, will be measured by gages conforming to Section 6-07.3(8)A.

Wet measurements will be taken immediately after the paint is applied in accordance with ASTM D4414. Dry measurements will be taken after the coating is dry and hard in accordance with SSPC Paint Application Specification No. 2.

Each painter shall be equipped with wet film thickness gages and shall be responsible for performing frequent checks of the paint film thickness throughout application.

Coating thickness measurements may be made by the Engineer after the application of each coat and before the application of the succeeding coat. In addition, the Engineer may inspect for uniform and complete coverage and appearance. One hundred percent of all thickness measurements shall meet or exceed the minimum wet film thickness. In areas where wet film thickness measurements are impractical, dry film thickness measurements may be made. If a question arises about an individual coat's thickness or coverage, it may be verified by the use of a Tooke gage in accordance with ASTM D4138.

If the specified number of coats does not produce a combined dry film thickness of at least the sum of the thicknesses required per coat, if an individual coat does not meet the minimum thickness, or if visual inspection shows incomplete coverage, the coating system will be rejected and the Contractor shall discontinue painting and surface preparation operations and shall submit a Type 2 Working Drawing of the repair proposal. The repair proposal shall include documentation demonstrating the cause of the less-than-minimum thickness, along with physical test results, as necessary, and modifications to Work methods to prevent similar results. The Contractor shall not resume painting or surface preparation operations until receiving the Engineer's acceptance of the completed repair.

6-07.3(9)E Environmental Condition Requirements Prior to Application of Paint

Paint shall be applied only during periods when:

1. Air and steel temperatures are in accordance with the paint manufacturer's recommendations but in no case less than 35°F nor greater than 115°F.
2. Steel surface temperature is a minimum of 5°F above the dew point.
3. Steel surface is not wet.
4. Relative humidity is within the manufacturer's recommended range.
5. The anticipated ambient temperature will remain above 35°F or the manufacturer's minimum temperature, whichever is greater, during the paint drying and curing period.

Application will not be allowed if conditions are not favorable for proper application and performance of the paint.

Paint shall not be applied when weather conditions are unfavorable to proper curing. If a paint system manufacturer's recommendations allow for application of a paint under environmental conditions other than those specified, the Contractor shall submit a Type 2 Working Drawing consisting of a letter from the paint manufacturer specifying the environmental conditions under which the paint can be applied. Application of paint under environmental conditions other than those specified in this section will not be allowed without the Engineer's concurrence.

6-07.3(9)F Shop Surface Cleaning and Preparation

A roughened surface profile shall be provided by an abrasive blasting procedure as accepted by the Engineer. The profile shall be 1-mil minimum or in accordance with the paint manufacturer's recommendations, whichever is greater. The entire steel surface to be painted, including surfaces specified in Section 6-07.3(9)G to receive a mist coat of primer, shall be cleaned to a near white condition in accordance with SSPC-SP 10, Near-white Metal Blast Cleaning, and shall be in this condition immediately prior to paint application.

6-07.3(9)G Application of Shop Primer Coat

After receiving the Engineer's acceptance of the prepared surface, the primer shall be applied so as to produce a uniform, even coating that has fully bonded with the metal. Primer shall be applied with the spray nozzles and pressures recommended by the manufacturer of the paint system, so as to attain the film thicknesses specified. Repairs of the shop primer coat shall be prepared in accordance with the painting plan. Shop primer coat repair paint shall be selected from the approved component based or performance based paint system in accordance with Section 6-07.3(10)H.

Steel girder top flanges and soldier pile flanges to be embedded in concrete shall be prepared in accordance with Section 6-07.3(9)F and shall then receive a mist coat of the specified primer with a dry film thickness of 0.5 to 1.0 mils.

The Contractor shall provide access to the steel to permit inspection by the Engineer. The access shall not mar or damage freshly painted surfaces.

High-strength field bolts shall not be painted before erection and installation.

6-07.3(9)H Containment for Field Coating

The Contractor shall use a containment system in accordance with Section 6-07.3(10)A for surface preparation and prime coating of all uncoated areas.

During painting operations of the intermediate, intermediate stripe and top coats the Contractor shall furnish, install, and maintain drip tarps below the areas to be painted to contain all spilled paint, buckets, brushes, and other deleterious material, and prevent such materials from reaching the environment below or adjacent to the structure being painted. Drip tarps shall be absorbent material and hung to minimize puddling. The Contractor shall evaluate the project-specific conditions to determine the specific type and extent of containment needed to control the paint emissions and shall submit a containment plan in accordance with Section 6-07.3(2).

6-07.3(9)I Application of Field Coatings

An on-site supervisor shall be present for each work shift at the bridge site.

Upon completion of erection Work, all uncoated or damaged areas remaining, including uncoated bolts, nuts, washers, and splice plates, shall be prepared in accordance with Section 6-07.3(9)F, followed by a field primer coat of a zinc-rich primer. Galvanized bolts, nuts and washers shall be prepared in accordance with Section 6-07.3(11)A. Final coats shall be applied over all primed or galvanized surfaces using paint selected from the approved component or performance based paint system in accordance with Section 6-07.3(10)H. The intermediate, intermediate stripe, and top coats shall be applied in accordance with the manufacturer's written recommendations.

Upon completion of erection Work, welds for steel column jackets may be prepared in accordance with SSPC-SP 15, Commercial Grade Power Tool Cleaning.

The minimum drying time between coats shall be as shown in the product data sheets, but not less than 12 hours. The Contractor shall determine whether the paint has cured sufficiently for proper application of succeeding coats.

The maximum time between intermediate and top coats shall be in accordance with the manufacturer's written recommendations. If the maximum time between coats is exceeded, all newly coated surfaces shall be prepared to SSPC-SP 7, Brush-off Blast Cleaning, and shall be repainted with the same paint that was cleaned, at no additional cost to the Contracting Agency.

Each coat shall be applied in a uniform layer, completely covering the preceding coat. The Contractor shall correct runs, sags, skips, or other deficiencies before application of succeeding coats. Such corrective work may require sanding, scraping, re-cleaning, application of additional paint, or other means as determined by the Engineer, at no additional cost to the Contracting Agency.

Dry film thickness measurements will be made in accordance with Section 6-07.3(9)D.

All paint damage that occurs shall be repaired in accordance with the manufacturer's written recommendations. On bare areas or areas of insufficient primer thickness, the repair shall include field-applied zinc-rich primer, and the final coats of the paint selected from the approved component or performance based paint system in accordance with Section 6-07.3(10)H. On areas where the primer is at least equal to the minimum required dry film thickness, the repair shall include the application of the final two coats of the paint system including stripe coats. All paint repair operations shall be performed by the Contractor at no additional cost or time to the Contracting Agency.

6-07.3(10) Painting Existing Steel Structures

Painting existing steel structures includes providing containment, cleaning, preparing the surface, painting metal surfaces, and disposal of generated waste. Painting of existing steel structures shall be done in the following sequence:

1. Containment.
2. Bird guano, fungus, and vegetation removal.
3. Dry cleaning.
4. Surface preparation.
5. Application of primer coat
6. Treatment of pack rust and gaps.
7. Complete paint system application.

6-07.3(10)A Containment

The containment system shall be in accordance with SSPC Technology Guide No. 6, *Guide for Containing Surface Preparation Debris Generated During Paint Removal Operations* Class 1. The containment system shall fully enclose the steel to be painted and not allow materials to escape the containment system. The Contractor shall protect the surrounding environment from all debris or damage resulting from the Contractor's operations.

Except as otherwise specified in the Contract, the containment length shall not exceed the length of a span (defined as pier to pier). The containment system shall not cause damage to the existing structure. Attachment devices shall not mark or otherwise damage the steel member to which they are attached. Field-welding of attachments to the existing structure will not be allowed. The Contractor shall not drill holes into the existing structure or through existing structural members except as shown in the Contractor's accepted painting plan Working Drawing submittal.

Emissions shall be assessed by Visible Emission Observations (Method A) in SSPC Technology Update No. 7, Conducting Ambient Air, Soil, and Water Sampling of Surface Preparation and Paint Disturbance Activities, Section 6.2 and shall be limited to the Level A Acceptance Criteria Option Level 0 Emissions standard. If visible emissions occur or if failure to the containment system occurs or if signs of failure to the containment system are present, the Contractor shall stop work immediately. Work shall not resume until the failure has been corrected to the satisfaction of the Engineer.

The containment system shall not be removed until all cleaned and painted surfaces have been inspected and accepted by the Engineer.

Prior to beginning work each day, all containment systems shall be inspected by the Contractor to verify they are in place and functioning properly. Maintenance required to restore full function shall be completed prior to beginning work.

6-07.3(10)B Bird Guano, Fungus, and Vegetation Removal

Bird guano and bird nesting materials shall be removed in the dry. Following dry removal, the Contractor shall apply a treatment solution in accordance with Section 9-08.5(1), followed by hand-scrubbing and rinsing with water in accordance with Section 9-08.5(3). The bird guano, bird nesting materials, and treatment solution shall be contained and collected.

Vegetation, soil, and other waste debris either intertwined with or coating the steel members included in the painting limits shall be contained, collected, removed, and disposed of in accordance with this Section and Section 6-01.12. When steel bearing assemblies are included within the painting limits, the Contractor shall also contain, collect, remove, and dispose of all vegetation, soil, and debris on the associated pier caps.

The Contractor shall treat all areas of fungus growth and vegetative growth. The Contractor shall apply a treatment solution in accordance with Section 9-08.5(2) to the fungus areas for a period recommended by the solution manufacturer or as specified by the Engineer, but in no case less than 5 minutes. The fungus, vegetative growth, and treatment solution shall be contained and collected.

Bird guano, bird nesting materials, fungus, and vegetative growth shall be disposed of at a land disposal site accepted by the Engineer. The Contractor shall submit a Type 1 Working Drawing consisting of the disposal receipt, which shall include a description of the disposed material.

6-07.3(10)C Dry Cleaning

Dry cleaning shall include removal of accumulated dirt and debris on the surfaces to be painted. Collected dirt and debris shall be disposed of at a land disposal site accepted by the Engineer. The Contractor shall submit a Type 1 Working Drawing consisting of the disposal receipt, which shall include a description of the disposed material. Documentation of the disposal shall comply with Section 1-07.5.

6-07.3(10)D Surface Preparation Prior to Overcoat Painting

The Contractor shall remove all visible oil, grease, and road tar in accordance with SSPC-SP 1, Solvent Cleaning.

Following preparation by SSPC-SP1, all steel surfaces to be painted shall be prepared in accordance with SSPC-SP 7, Brush-Off Blast Cleaning. Surfaces inaccessible to brush-off blast shall be prepared in accordance with SSPC-SP 3, Power Tool Cleaning, as allowed by the Engineer.

Following brush-off blast cleaning, the Contractor shall perform spot abrasive blast cleaning in accordance with SSPC-SP 6, Commercial Blast Cleaning. Spot abrasive blast cleaning shall be performed in such a manner that the adjacent areas of work are protected from damage. Areas exhibiting coating failure down to the steel substrate, those exhibiting visible corrosion, and those areas needing treatment of pack rust and gaps shall be prepared down to clean bare steel in accordance with SSPC-SP 6. Exposed steel areas that have an average exposed diameter of less than 1½ inches and no other similar area closer than 4 inches do not require spot abrasive blast cleaning or edge feathering unless required by the Engineer. The Contractor shall provide a sharp angular surface profile by an abrasive blasting procedure as accepted by the Engineer. The profile shall be 1 mil minimum or in accordance with the paint manufacturer's recommendations, whichever is greater. For small areas, as allowed by the Engineer, the Contractor may substitute cleaning in accordance with SSPC-SP 15, Commercial Grade Power Tool Cleaning. The prepared area shall extend at least 2 inches into adjacent tightly adhering, intact coating.

Following spot abrasive blast cleaning of exposed steel surfaces, edges of tightly adherent coating remaining shall be feathered so that the recoated surface has a smooth appearance. Immediately prior to painting, the Contractor shall clean all steel surfaces and staging areas with dry, oil-free compressed air conforming to ASTM D4285.

6-07.3(10)E Surface Preparation – Full Paint Removal

For structures where full removal of existing paint is specified, the Contractor shall remove all visible oil, grease, and road tar in accordance with SSPC-SP 1.

Following preparation by SSPC-SP 1, all steel surfaces to be painted, and all areas needing treatment of pack rust and gaps, shall be prepared in accordance with SSPC-SP 10, Near-White Metal Blast Cleaning. Surfaces inaccessible to Near-White Metal Blast Cleaning shall be prepared in accordance with SSPC-SP 11, Power Tool Cleaning to Bare Metal, as allowed by the Engineer.

6-07.3(10)F Collecting, Testing, and Disposal of Containment Waste

The sealed waste containers shall be labeled as required by State and Federal laws. All confined materials shall be collected and secured in sealed containers at the end of each shift or daily at a minimum to prevent the weight of the confined materials from causing failure to the containment system. The sealed waste containers shall be stored in accordance with Section 1-06.4, the painting plan, and the following requirements:

1. The containers shall be stored on an impermeable surface that accommodates sweeping or vacuuming.
2. Landside storage of the containers shall be at an elevation above the ordinary high water level (OHWL) elevation. The container storage area shall not be in a stormwater runoff course and shall not be in an area of standing water.
3. The container storage area shall be a fenced, secured site, separate from the storage facilities for paint materials and paint equipment.
4. The containers shall not be stored at the on-site landside storage site for longer than 90 calendar days.

All material collected by and removed from the containment system shall be taken to a landside staging area, provided by the Contractor, for further processing and storage prior to transporting for disposal. Handling and storage of material collected by and removed from the containment system shall conform to Section 1-06.4. Storage of containment waste materials shall be in a facility separate from the storage facilities used for paint materials and paint equipment.

Containment waste is defined as all paint chips and debris removed from the steel surface and all abrasive blast media, as contained by the containment system. After all waste from the containment system has been collected, the Contractor shall collect representative samples of the components that field screening indicates are lead-contaminated material. The Contractor shall collect at least one representative sample from each container. The Contractor may choose to collect a composite sample of each container, but the composite sample must consist of several collection points (a minimum of 3 random samples) that are representative of the entire contents of the container and representative of the characteristics of the type of waste in the container. In accordance with WAC 173–303-040, a representative sample means “a sample which can be expected to exhibit the average properties of the sample source.”

The debris shall be tested for metals using the Toxicity Characteristics Leaching Procedure (TCLP) and EPA Methods 1311 and 6010. At a minimum, the materials should be analyzed for the Resource Conservation and Recovery Act (RCRA) 8 Metals (arsenic, barium, cadmium, chromium, lead, mercury, selenium, and silver). Pursuant to the Dangerous Waste (DW) Regulations Chapter 173-303-90(8)(c) WAC, waste that contains contaminants that occur at concentrations at or above the DW threshold must be designated as DW. All material within each individual container or containment system that designates as DW shall be disposed of at a legally permitted Subtitle C Hazardous Waste Landfill. All material within each individual container or containment system that designate below the DW threshold, will be designated as “Solid Waste” and shall be disposed of at a legally permitted Subtitle D Landfill. Disposal shall be in accordance with WAC 173-303 for waste designated “Dangerous Waste” and pursuant to WAC 173-350 for waste designated as “Solid Waste”.

The Contractor shall submit a Type 1 Working Drawing consisting of the transmittal documents or bill of lading listing the waste material shipped from the construction site to the waste disposal site. Both the copy of the shipment list showing the signature of the Engineer and the copy showing the waste site operator’s confirmation for receipt of the waste shall be submitted. Documentation of the disposal shall comply with Section 1-07.5.

In the event that the containment wastes are designated as “Dangerous Wastes” or “Extremely Hazardous Waste” under WAC 173-303, the Contracting Agency will provide to the Contractor the appropriate EPA identification number.

Unless noted otherwise, a waste site will not be provided by the Contracting Agency for the disposal of excess materials and debris.

The Contractor shall submit a Type 1 Working Drawing of all TCLP results.

The Contractor shall submit a Type 1 Working Drawing consisting of waste disposal documentation within 15 working days of each disposal. This documentation shall include the quantity and type of waste disposed of with each disposal shipment.

6-07.3(10)G Treatment of Pack Rust and Gaps

Pack rust is defined as the condition where two or more pieces of steel fastened together by rivets or bolts have been pressed apart by crevice corrosion caused by the buildup of corrosion products at the interface of the steel pieces.

Pack rust forming a gap between steel surfaces of 1⁄16 to ¼ inch shall be cleaned to a depth of at least one half of the gap width. The gaps shall be cleaned and prepared in accordance with Section 6-07.3(10)D or 6-07.3(10)E as specified in the Contract. When cleaned and prepared in accordance with Section 6-07.3(10)D, the cleaned gap shall be treated with rust penetrating sealer, prime coated, and then caulked. When cleaned and prepared in accordance with Section 6-07.3(10)E, the cleaned gap shall be primed and then treated with rust penetrating sealer, and then caulked. The caulking shall be applied to form a watertight seal along the top edge and the two sides of the steel pieces involved, using the rust penetrating sealer and caulk as accepted by the Engineer. The bottom edge or lowest edge of the steel pieces involved shall not be caulked.

The type of rust penetrating sealer and caulk used shall be compatible with the paint system used and shall be applied in accordance with the rust penetrating sealer and caulk manufacturer’s instructions. Caulk shall be a single-component polyurethane sealant conforming to Section 9-08.7.

When caulking joints where only one steel piece edge is exposed, a fillet of caulk shall be formed that is not less than ⅛ inch or the width of the pack rust gap. The fillet is not required where there is no separation of the steel pieces due to pack rust.

At locations where gaps between steel surfaces exceed ¼ inch after cleaning, preparing, sealing and priming, the Contractor shall then fill the gap with foam backer rod material as accepted by the Engineer. The foam backer rod material shall be of sufficient diameter to fill the crevice or gap. The Contractor shall apply caulk over the foam backer rod material to form a watertight seal.

Caulk and backer rod, if needed, shall be placed prior to applying the top coat.

6-07.3(10)H Paint System

The paint system applied to existing steel surfaces shall consist of the following five-coat system:

Option 1 (component based system):

Primer Coat - Zinc-rich Moisture Cured Polyurethane	9-08.1(2)F
Primer Stripe Coat - Moisture Cured Polyurethane	9-08.1(2)F
Intermediate Coat - Moisture Cured Polyurethane	9-08.1(2)G
Intermediate Stripe Coat - Moisture Cured Polyurethane	9-08.1(2)G
Top Coat - Moisture Cured Polyurethane	9-08.1(2)H

Option 2 (performance based system):	
Primer Coat - Zinc-rich Epoxy	9-08.1(2)N
Primer Stripe Coat - Epoxy	9-08.1(2)N
Intermediate Coat - Epoxy	9-08.1(2)N
Intermediate Stripe Coat - Epoxy	9-08.1(2)N
Top Coat - Polyurethane	9-08.1(2)N

Paints and related materials shall be a product listed in the current WSDOT Qualified Products List (QPL). Component based paint systems shall be listed on the QPL in the applicable sections of Section 9-08. Performance based systems shall be listed on the current Northeast Protective Coatings Committee (NEPCOAT) Qualified Products List "B" as listed on the WSDOT QPL in Section 9-08.1(2)N. If the paint and related material for the component based system is not listed in the current WSDOT QPL, a sample shall be submitted to the State Materials Laboratory in Tumwater for evaluation and acceptance in accordance with Section 9-08.

All paint coating components of the selected paint system shall be produced by the same manufacturer. Only one paint system from a singular manufacturer shall be used throughout the project unless otherwise allowed in writing by the Engineer. The Contractor shall not change to a different paint system once the initial paint system has been applied to any portion of the bridge unless otherwise allowed in writing by the Engineer.

6-07.3(10)I Paint Color

Each of the five coats shall be a contrasting color to the previously applied full coat. The color of the top coat shall be as specified in the Plans or Special Provisions and shall conform to Section 9-08.1(8). Tinting shall occur at the factory at the time of manufacture and placement in containers, prior to initial shipment. Application site tinting will not be allowed except as otherwise allowed by the Engineer.

6-07.3(10)J Mixing and Thinning Paint

Mixing and thinning paint shall be in accordance with Section 6-07.3(9)C.

6-07.3(10)K Coating Thickness

Coating thickness shall be in accordance with Section 6-07.3(9)D. The minimum dry film thickness of the combination of intermediate and intermediate stripe coat, shall not be less than 3.0 mils.

6-07.3(10)L Environmental Condition Requirements Prior to Application of Paint

Environmental conditions shall be in accordance with Section 6-07.3(9)E.

6-07.3(10)M Steel Surface Condition Requirements Prior to Application of Paint

The steel surface to be painted shall be free of moisture, dirt, dust, grease, oil, loose, peeling or, chalky paint, abrupt paint edges, salts, rust, mill scale, and other foreign matter and substances that would prevent the bond of the succeeding application. The Contractor shall protect freshly painted surfaces from contamination by abrasives, dust, or foreign materials from any other source. The Contractor shall prepare contaminated surfaces to the satisfaction of the Engineer before applying additional paint.

Prepared surfaces shall be kept clean at all times, before painting and between coats.

Edges of existing paint shall be feathered in accordance with SSPC-PA 1, Shop, Field, and Maintenance Coating of Metals, Note 15.20.

6-07.3(10)N Field Coating Application Methods

The Contractor shall apply paint materials in accordance with manufacturer's recommendations by air or airless spray, brush, roller, or any combination of these methods unless otherwise specified. Spray application of the paint shall be accomplished with spray nozzles and at pressures as recommended by the paint manufacturer to ensure application of paint at the specified film thickness. The Contractor may apply stripe coat paint using spray or brush but shall follow spray application using a brush to ensure complete coverage around structural geometric irregularities and to push the paint into gaps between existing steel surfaces and around rivets and bolts. All application techniques shall conform to Section 7, SSPC-PA 1. Painters using brushes shall work from pails containing a maximum of 2 gallons of paint.

6-07.3(10)O Applying Field Coatings

An on-site supervisor shall be present for each work shift at the bridge site.

The first coat shall be a primer coat applied to steel surfaces cleaned to bare metal. The second coat shall be a primer stripe coat applied to all steel surfaces cleaned to bare metal and defined to receive a stripe coat. The third coat shall be an intermediate coat. The fourth coat shall be an intermediate stripe coat applied to steel surfaces defined to receive a stripe coat. The fifth coat shall be the top coat. The intermediate (third) and top (fifth) coats shall encapsulate the entire surface area of the structure members specified to be painted.

Prior to the application of paint, the Contractor shall clean the bridge deck surface for the purpose of dust control. All expansion or contraction joints specified in the plans to be sealed shall be sealed prior to application of the top coat. All rust staining or debris on steel surfaces, below or in the vicinity of the joints, shall be cleaned prior to application of the top coat at no additional expense to the Contracting Agency.

During painting operations the Contractor shall furnish, install, and maintain drip tarps below the areas to be painted to contain all spilled paint, buckets, brushes, and other deleterious material, and prevent such materials from reaching the environment below or adjacent to the structure being painted. Drip tarps shall be absorbent material and hung to minimize puddling.

In addition to the requirements of the Specifications, paint application shall conform to:

1. The best practices of the trade.
2. The written recommendations of the paint manufacturer.
3. All applicable portions of the SSPC-PA 1.

No primer paint shall be applied to surfaces until the surface has been inspected and accepted by the Engineer. Areas to which primer paint has been applied without the Engineer's inspection and acceptance will be considered improperly cleaned. The unauthorized application shall be completely removed and the entire area recleaned to the satisfaction of the Engineer. After the area has been recleaned, inspected, and approved, the Contractor may again initiate the painting sequence. No additional compensation or extension of time in accordance with Section 1-08.8 will be allowed for the removal of unauthorized paint application and recleaning of the underlying surface.

All steel surfaces cleaned to bare metal by abrasive blast cleaning shall receive the primer coat within the same working day as the cleaning to bare metal and before any rust begins to form. Each successive coat shall be applied as soon as possible over the previous coat, accounting for drying time of the preceding coat, weather, atmospheric temperature and other environmental conditions, and the paint manufacturer's recommendations. Each coat shall be dry before recoating and shall be sufficiently cured so that succeeding or additional coats may be applied without causing damage to the previous coat. Recoat times shall be as shown in the paint manufacturer's recommendations, but not less than

12 hours. Revision of recoat times to other than recommended by the paint manufacturer requires the concurrence of the Engineer. If the maximum time between coats is exceeded, all affected areas shall be prepared to SSPC-SP 7, Brush-off Blast Cleaning, and recoated with the Contract-specified system at no additional expense or time to the Contracting Agency.

Each coat shall be applied in a uniform layer, completely covering the preceding coat. The Contractor shall correct runs, sags, skips, or other deficiencies before application of succeeding coats. Such corrective work may require sanding, scraping, recleaning, application of additional paint, or other means in accordance with the paint manufacturer's recommendations and as determined by the Engineer, at no additional expense or time to the Contracting Agency.

If fresh paint is damaged by the elements, the Contractor shall replace or repair the paint to the satisfaction of the Engineer at no additional cost to the Contracting Agency.

After applying the primer or intermediate coats, the Contractor shall apply a primer or intermediate stripe coat, respectively, on all edges, corners, seams, crevices, interior angles, junction of joint members, rivet or bolt heads, nuts and threads, weld lines, and other irregular surfaces. The coverage of each stripe coat shall extend at least 1 inch beyond the irregular surface. The stripe coat shall be of sufficient thickness to completely hide the surface being covered and shall be followed as soon as feasible by the application of the subsequent coat to its specified thickness.

Each application of primer, primer stripe, intermediate, intermediate stripe, and top coat shall be considered as separately applied coats. The Contractor shall not use a preceding or subsequent coat to remedy a deficiency in another coat. The Contractor shall apply the top coat to at least the minimum specified top coat thickness, to provide a uniform appearance and consistent finish coverage.

If roadway or sidewalk planks lie so close to the metal that they prevent proper cleaning and painting, the Contractor shall remove or cut the planks to provide at least a 1-inch clearance. Any plank removal or cutting shall be done with the concurrence of the Engineer. The Contractor shall replace all planks after painting. If removal breaks or damages the planks and makes them unfit for reuse, the Contractor shall replace them at no expense to the Contracting Agency.

6-07.3(10)P Field Coating Repair

Paint repair shall conform to SSPC-PA 1. Repair areas shall be cleaned of all damaged paint and the system reapplied using all coats typical to the paint system and shall meet the minimum coating thickness. Each coat shall be thoroughly dry before applying subsequent coats. Paint repair shall be in accordance with the paint manufacturer's recommendations and as accepted by the Engineer.

6-07.3(10)Q Cleanup

Cleaning of equipment shall not be done in State waters nor shall resultant cleaning runoff be allowed to enter State waters. No paint cans, lids, brushes, or other debris shall be allowed to enter State waters. Solvents, paints, paint sludge, cans, buckets, rags, brushes, and other waste associated with this project shall be collected and disposed of off-site. Paint products, petroleum products, or other deleterious material shall not be wasted into, or otherwise enter, State waters as a result of project activities.

Cleanup of the project site shall conform to Sections 1-04.11 and 6-01.12

6-07.3(11) Painting or Powder Coating of Galvanized Surfaces

Galvanized surfaces specified to be coated after galvanizing shall receive either paint in accordance with Section 6-07.3(11)A or powder coating in accordance with Section 6-07.3(11)B. The color of the finish coat shall be as specified in the Special Provisions.

6-07.3(11)A Painting of Galvanized Surfaces

All galvanized surfaces receiving paint shall be prepared for painting in accordance with the ASTM D6386. The method of preparation shall be brush-off blast in accordance with SSPC-SP16 Brush-Off Blast Cleaning of Coated and Uncoated Galvanized Steel, Stainless Steels, and Non-Ferrous Metals or as otherwise allowed by the Engineer. The Contractor shall not begin painting until receiving the Engineer's acceptance of the prepared galvanized surface. For galvanized bolts used for replacement of deteriorated existing rivets or for new painted steel structures, and for small areas less than one square foot, the Contractor, with the concurrence of the Engineer and after successful demonstration testing, may prepare galvanized surfaces in accordance with SSPC-SP1, Solvent Cleaning followed by SSPC-SP2, Hand Tool Cleaning or SSPC-SP3, Power Tool Cleaning. The demonstration testing shall include adhesion testing of the first coat of paint over galvanized bolts, nuts, and washers or a representative galvanized surface. Adhesion testing shall be performed in accordance with ASTM D4541 for 600 psi minimum adhesion. A minimum of 3 successful tests shall be performed on the galvanized surface prepared and painted using the same methods and materials to be used on the galvanized bolts, nuts and washers in the field.

6-07.3(11)A1 Environmental Conditions

Steel surfaces shall be:

- Greater than 35°F, and
- Less than 115°F.

or in accordance with the manufacturer's recommendations, whichever is more stringent.

6-07.3(11)A2 Paint Coat Materials

The Contractor shall paint the dry surface as follows:

1. The first coat over a galvanized surface shall be an epoxy polyamide conforming to Section 9-08.1(2)E. Alternatively, an intermediate coat conforming to Section 9-08.1(2)G or Section 6-07.3(10)H Option 2 NEPCOAT performance based paint specification may be used as a first coat provided it passes the adhesion testing specified in Section 6-07.3(11)A. The first coat shall be compatible with galvanizing and as recommended by the top coat manufacturer.
2. The second coat shall be a top coat moisture cured aliphatic polyurethane conforming to Section 9-08.1(2)H or a top coat polyurethane conforming to Section 6-07.3(10)H Option 2 NEPCOAT performance based paint specification compatible with the first coat as recommended by the manufacturer.

Each coat shall be dry before the next coat is applied. All coats applied in the shop shall be dried hard before shipment.

6-07.3(11)B Powder Coating of Galvanized Surfaces

Powder coating of galvanized surfaces shall consist of the following coats:

1. The first coat shall be an epoxy powder primer coat conforming to Section 9-08.2.
2. The second coat shall be a polyester finish coat conforming to Section 9-08.2.

6-07.3(11)B1 Submittals

The Contractor shall submit Type 2 Working Drawings consisting of the following information. The submittal shall be assembled in an order consistent with the submittal requirements below:

1. The name, location, and contact information (mail address, phone, and email) for the firm performing the powder coating operation.
2. Project title and contract number information

3. Project-specific powder coating plan including as a minimum the following:
    a. Identification of powder coating materials and manufacturer
    b. Cleaning and surface preparation
    c. Preheating and powder coating application process
    d. Curing process and durations
    e. Powder coating manufacturer's recommended shop and field coating repair plan including repair materials
    f. Handling, storage, and shipping process
4. Quality control (QC) program plan including as a minimum the following:
    a. Quality control and inspection procedures
    b. Inspection hold points
    c. Forms to document inspection and testing of the coatings
5. Product data and SDS sheets for all powder coating and coating repair materials.

6-07.3(11)B2 Galvanizing

Prior to the galvanizing operation, the Contractor shall identify to the galvanizer the specific assemblies and surfaces receiving the powder coating after galvanizing, to ensure that the galvanizing method used on these assemblies is compatible with subsequent application of a powder coating system. Specifically, such assemblies shall neither be water-quenched nor receive a chromate conversion coating as part of the galvanizing operation.

6-07.3(11)B3 Galvanized Surface Cleaning and Preparation

Galvanized surfaces receiving the powder coating shall be cleaned and prepared for coating in accordance with ASTM D7803, and the project-specific powder coating plan.

Assemblies conforming to the ASTM D7803 definition for newly galvanized steel shall receive surface smoothing and surface cleaning in accordance with ASTM D7803, Section 5, and surface preparation in accordance with ASTM D7803, Section 5.1.3.1.

Assemblies conforming to the ASTM D7803 definition for partially weathered galvanized steel shall be checked and prepared in accordance with ASTM D7803, Section 6, before then receiving surface smoothing and surface cleaning in accordance with ASTM D7803, Section 5, and surface preparation in accordance with ASTM D7803, Section 5.1.3.1.

Assemblies conforming to the ASTM D7803 definition for weathered galvanized steel shall be prepared in accordance with ASTM D7803, Section 7 before then receiving surface smoothing and surface cleaning in accordance with ASTM D7803, Section 5, and surface preparation in accordance with ASTM D7803, Section 5.3.

Ferrous metal abrasives are prohibited as a blast media for surface preparation.

Surface preparation shall be accomplished using dry abrasive blasting through a blast nozzle with compressed air. Abrasive blasting with a centrifugal wheel is prohibited.

The Contractor shall notify the Engineer of all surface cleaning and preparation activities and shall provide the Engineer opportunity to perform quality assurance inspection, in accordance with Section 1-05.6, at the completion of surface cleaning and preparation activities prior to beginning powder coating application.

6-07.3(11)B4 Powder Coating Application and Curing

After surface preparation, the two-component powder coating shall be applied in accordance with the powder coating manufacturer's recommendations, the project-specific powder coating plan, and as follows:

1. Preheat. The preheat shall be sufficient to prevent pinholes from forming in the finished coating system.
2. Apply the epoxy primer coat, followed by a partial cure.
3. Apply the polyester finish coat, followed by the finish cure.

6-07.3(11)B5 Testing

The firm performing the powder coating operation shall conduct, or make arrangements for, QC testing on all assemblies receiving powder coating for this project, in accordance with the powder coating firm's QC program as documented in item 4 of the Submittal Subsection above. Testing may be performed on coated surfaces of production fabricated items, or on a representative test panel coated alongside the production fabricated items being coated. There shall be a minimum of one set of tests representing each cycle of production fabricated items coated and cured. Additional tests shall be performed at the request of the Engineer. Repair of damaged coatings on production fabricated items shall be the responsibility of the firm applying the powder coating, and shall be in accordance with the project-specific powder coating plan. At a minimum, the QC testing shall test for the following requirements:

1. Visual inspection for the presence of coating holidays and other unacceptable surface imperfections.
2. Coating thickness measurement in accordance with Section 6-07.3(5). The minimum thickness of the epoxy primer coating and polyester finish coating shall be 3 mils each.
3. Hardness testing in accordance with ASTM D3363, with the finish coat providing a minimum hardness value of H.
4. Adhesion testing in accordance with ASTM D4541 for 600 psi minimum adhesion for the complete two-component coating system.
5. Powder Coating Institute (PCI) #8 recommended procedure for solvent cure test.

The results of the QC testing shall be documented in a QC report and submitted as a Type 2 Working Drawing.

The Engineer shall be provided notice and access to all assemblies at the powder coating facility for the purposes of Contracting Agency acceptance inspection, including notice and access to witness all hardness and adhesion testing performed by the firm conducting the QC testing, in accordance with Section 1-05.6.

Assemblies not meeting the above requirements will be subject to rejection by the Engineer. Rejected assemblies shall be repaired or recoated by the Contractor, at no additional expense to the Contracting Agency, in accordance with the powder coating manufacturer's recommendation as detailed in the project-specific powder coating plan, until the assemblies satisfy the acceptance testing requirements.

Assemblies shall not be shipped from the powder coating firm's facility to the project site until the Contractor receives the Engineer's acceptance of the QC Report and assembly inspection performed by the Engineer.

6-07.3(11)B6 Coating Protection for Shipping, Storage, and Field Erection

After curing and acceptance, the Contractor shall protect the coated assemblies with multiple layers of bubble wrap or other protective wrapping materials specified in the project-specific powder coating plan.

During storage and shipping, each assembly shall be separated from other assemblies by expanded polystyrene spacers and other spacing materials specified in the project-specific powder coating plan.

After erection, all coating damage due to the Contractor's shipping, storage, handling, and erection operations shall be repaired by the Contractor in accordance with the project-specific powder coating plan. The Contractor shall provide the Engineer access to all locations of all powder-coated members for verification of coating conditions prior to and following all coating repairs.

6-07.3(12) Painting Ferry Terminal Structures

Painting of ferry terminal structures shall be in accordance with Section 6-07.3 as supplemented below.

6-07.3(12)A Painting New Steel Ferry Terminal Structures

Painting of new steel Structures shall be in accordance with Section 6-07.3(9) except that all coatings (primer, intermediate, intermediate stripe, and top) shall be applied in the shop with the following exceptions:

1. Steel surfaces to be field welded.
2. Steel surfaces to be greased.
3. The length of piles designated in the Plans not requiring painting.

The minimum drying time between coats shall be as shown in the product data sheets, but not less than 12 hours. The Contractor shall determine whether the paint has cured sufficiently for proper application of succeeding coats.

6-07.3(12)A1 Paint Systems

Paint systems for Structural Steel, which includes vehicle transfer spans and towers, pedestrian overhead loading structures and towers, upland structural steel and other elements as designated in the Special Provisions shall be as specified in Section 6-07.3(9)A.

Paint systems for Piling, Landing Aids and Life Ladders shall be as specified in the Special Provisions.

6-07.3(12)A2 Paint Color

Paint colors shall be as specified in the Special Provisions.

6-07.3(12)A3 Coating Thickness

Coating thicknesses shall be as specified in the Special Provisions.

6-07.3(12)A4 Application of Field Coatings

An on-site supervisor shall be present for each work shift at the project site.

Upon completion of erection Work, all uncoated or damaged areas remaining, including bolts, nuts, washers, splice plates, and field welds shall be prepared in accordance with SSPC-SP 1, Solvent Cleaning, followed by SSPC-SP 11, Power Tool Cleaning to Bare Metal. Surface preparation shall be measured according to SSPC-VIS 3. SSPC-SP 11 shall be performed for a minimum distance of 1 inch from the uncoated or damaged area. In addition, intact shop-applied coating surrounding the area shall be abraded or sanded for a distance of 6 inches out from the properly prepared clean/bare metal areas to provide adequate roughness for application of field coatings. All sanding dust and contamination shall be removed prior to application of field coatings.

Field applied paint for Structural Steel shall conform to Section 6-07.3(10)H, as applicable. Field applied paint for Piling, Landing Aids and Life Ladders shall be as specified in the Special Provisions.

For areas above the tidal zone, the minimum drying time between coats shall be as shown in the product data sheets, but not less than 12 hours. For areas within the tidal zone, the minimum drying time between coats shall be as recommended by the paint system manufacturer. The Contractor shall determine whether the paint has cured sufficiently for proper application of succeeding coats.

The maximum time between intermediate and top coats shall be in accordance with the manufacturer's written recommendations. If the maximum time between coats is exceeded, all newly coated surfaces shall be prepared to SSPC-SP 3, Power Tool Cleaning, and shall be repainted with the same paint that was cleaned, at no additional cost to the Contracting Agency.

Each coat shall be applied in a uniform layer, completely covering the preceding coat. The Contractor shall correct runs, sags, skips, or other deficiencies before application of succeeding coats. Such corrective work may require re-cleaning, application of additional paint, or other means as determined by the Engineer, at no additional cost to the Contracting Agency.

Surface preparation for underwater locations shall consist of removing all dirt, oil, grease, loose paint, loose rust, and marine growth from the area that is to be repaired. The sound paint surrounding the damaged area shall be roughened to meet the requirements of the manufacturer. Paint for underwater applications shall be as specified in the Special Provisions and shall be applied in accordance with the manufacturer's recommendations.

6-07.3(12)B Painting Existing Steel Ferry Terminal Structures

Painting of existing steel structures shall be in accordance with Section 6-07.3(10) as supplemented by the following.

6-07.3(12)B1 Containment

Containment for full removal shall be in accordance with Section 6-07.3(10)A. Containment for overcoat systems shall be in accordance with all applicable Permits as required in the Special Provisions.

Prior to cleaning the Contractor shall enclose all exposed electrical and mechanical equipment to seal out dust, water, and paint. Non-metallic surfaces shall not be abrasive blasted or painted. Unless otherwise specified, the following metallic surfaces shall not be painted and shall be protected from abrasive blasting and painting:

1. Galvanized and stainless steel surfaces not previously painted,
2. Non-skid surfaces,
3. Unpainted intentionally greased surfaces,
4. Equipment labels, identification plates, tags, or other identifying markings,
5. Fire and emergency containers or boxes,
6. Mechanical hardware such as hoist sheaves, hydraulic cylinders, gear boxes, wire rope, or other hardware identified by the Engineer.

The Contractor shall submit a Type 2 Working Drawing consisting of materials and equipment used to shield components specified to not be cleaned and painted.

The Contractor shall shut off the power prior to working around electrical equipment. The Contractor shall follow the lock-out/tag-out safety provisions of the WAC 296-803 and all other applicable safety standards.

6-07.3(12)B2 Surface Preparation

For applications above high water and within the tidal zone, surface preparation for overcoat painting shall be in accordance with SSPC-SP 1, Solvent Cleaning, followed by SSPC-SP 3, Power Tool Cleaning. Use of wire brushes is not allowed. After SP 3 cleaning has been completed all surfaces exhibiting coating failure down to the steel substrate, and those exhibiting visible corrosion, shall be prepared down to clean bare steel in accordance with SSPC-SP 15, Commercial Grade Power Tool Cleaning. Surface preparation shall be measured according to SSPC-VIS 3. SSPC-SP 15 shall be performed for a minimum distance of 1 inch from the area exhibiting failure or visible corrosion. In addition, intact shop-applied coating surrounding the repair area shall be abraded or sanded for a distance of 6 inches out from the properly prepared clean/bare metal areas to provide adequate roughness for application of repair coatings. All sanding dust and contamination shall be removed prior to application of repair coatings. Surface preparation for full paint removal shall be in accordance with Section 6-07.3(10)E except SSPC-SP 11 will be permitted as detailed in the Contractor's painting plan and as allowed by the Engineer.

Surface preparation for underwater locations shall consist of removing all dirt, oil, grease, loose paint, loose rust, and marine growth from the area that is to be repaired. The sound paint surrounding the damaged area shall be roughened as required by the coating manufacturer.

Removed marine growth may be released to state waters provided the marine growth is not mixed with contaminants (paint, oil, rust, etc.) and it shall not accumulate on the sea bed. All marine growth containing contaminants shall be collected for proper disposal.

Surface preparation for the underside of bridge decks (consisting of either a steel grid system of main bars or tees and a light gauge metal form, in-filled with concrete or a corrugated light gauge metal form, infilled with concrete) shall be in accordance with SSPC-SP 2, Hand Tool Cleaning or SSPC-SP 3, Power Tool Cleaning with the intent of not causing further damage to the light gauge metal form. Following removal of pack rust and corroded sections from the underside of the bridge deck, cleaning and flushing to remove salts and prior to applying the primer coat, the Contractor shall seal the entire underside of the deck system with rust-penetrating sealer. Damage to galvanized metal forms and/or grids shall be repaired in accordance with ASTM A 780, with the preferred method of repair using paints containing zinc dust.

6-07.3(12)B3 Paint Systems

Paint systems for Structural Steel, which includes vehicle transfer spans and towers, pedestrian overhead loading structures and towers, upland structural steel and other elements as designated in the Special Provisions shall be as specified in Section 6-07.3(10)H.

Paint systems for Piling, Landing Aids, Life Ladders, underside of vehicle transfer span bridge decks, non-skid surface treated areas, and anti-graffiti coatings shall be as specified in the Special Provisions.

6-07.3(12)B4 Paint Color

Paint colors shall be as specified in the Special Provisions.

6-07.3(12)B5 Coating Thickness

Coating thicknesses shall be as specified in the Special Provisions.

6-07.3(12)B6 Application of Field Coatings

Application of field coatings shall be in accordance with Section 6-07.3(10)O and Section 6-07.3(12)A4 except for the following:

1. All coatings applied in the field shall be applied using a brush or roller. Spray application methods may be used if allowed by the Engineer.
2. Applied coatings shall not be immersed until the coating has been cured as required by the coating manufacturer.
3. Non-skid surface treatment products shall be applied in accordance with the manufacturer's recommendations.
4. Anti-graffiti coatings shall be applied in one coat following application of the top coat, where specified in the Plans.

6-07.3(13) Painting Timber Structures

Timber structures shall be painted as specified in the Special Provisions.

6-07.3(14) Metallic Coatings

6-07.3(14)A General Requirements

This specification covers the requirements for thermal spray metallic coatings, with and without additional paint coats, as a means to prevent corrosion.

The coating system consists of surface preparation by wash cleaning and abrasive blast cleaning, thermal spray application of a metallic coating using a material made specifically for that purpose, and, when specified, shop primer coat or shop primer coat plus top coat in accordance with Section 6-07.3(11)A. The system also includes inspection and acceptance requirements.

6-07.3(14)B Reference Standards

SSPC-SP 10/NACE No. 2	Near-White Blast Cleaning
SSPC CS 23.00	Specification for the Application of Thermal Spray Coatings (Metallizing) of Aluminum, Zinc, and Their Alloys and Composites for the Corrosion Protection of Steel
ASTM C633	Standard Test Method for Adhesion or Cohesion Strengths of Thermal Spray Coatings
ASTM D4417	Standard Test Methods for Field Measurement of Surface Profile of Blast-Cleaned Steel
ASTM D6386	Standard Practice for Preparation of Zinc (Hot-Dip Galvanized) Coated Iron and Steel Product and Hardware Surfaces for Painting
ASTM D4541	Standard Test Method for Pull-Off Strength of Coatings Using Portable Adhesion Testers
ANSI/AWS C2.18	Guide for the Protection of Steel with Thermal Sprayed Coatings of Aluminum, Zinc and their Alloys and Composites

6-07.3(14)C Quality Assurance

A representative sample of each lot of the coating material used shall be submitted to the Engineer for analysis prior to use. Zinc shall have a minimum purity of 99.9 percent. Zinc Aluminum 85/15 wire shall be 14 to 16 percent maximum aluminum.

The thermal sprayed coating shall have a uniform appearance. The coating shall not contain blisters, cracks, chips or loosely adhering particles, oil or other surface contaminants, nodules, or pits exposing the substrate.

The thermal spray coating shall adhere to the substrate with a minimum bond of 700 psi. The Contractor's QA program shall include thermal spray coating bond testing.

The Engineer may cut through the coating with a knife or chisel. If upon doing so, any part of the coating lifts away from the base metal ¼ inch or more ahead of the cutting blade without cutting the metal, then the bond is considered not effective and is rejected.

Coated areas that have been rejected or damaged in the inspection procedure described shall have the defective sections blast cleaned to remove all of the thermal sprayed coating and shall then be recoated. Before resubmittal and inspection, those sections where coating has not reached the required thickness shall be sprayed with additional metal until that thickness is achieved.

6-07.3(14)D Submittals

The Contractor shall submit to the Engineer, prior to abrasive blast cleaning, a 12 inch square steel plate, of the same material and approximate thickness of the steel to be coated, blasted clean in accordance with Section 6-07.3(14)E. The sample plate will be checked for specified angular surface pattern, the abrasive grit size and type used, and the procedure used. This plate shall be used as the visual standard to determine the acceptability of the cleaned surface. In the event the Contractor's cleaning operation is inferior to the sample plate, the Contractor shall be required to correct the cleaning operation to do a job comparable to the specimen submitted.

At the same time as submitting the abrasive blast cleaned steel plate sample, the Contractor shall submit to the Engineer, a second 12 inch square steel plate of the same material and thickness, cleaned and thermal spray coated in accordance with the same processes and with the same equipment as intended for use in applying the thermal spray coatings. The Engineer may request additional cleaned and thermal spray coated samples to be produced and submitted coincident with thermal spray coating of the items specified in the Plans to receive thermal spray coatings.

6-07.3(14)E Surface Preparation

Surface irregularities (e.g., sharp edges and/or carburized edges, cracks, delaminations, pits) interfering with the application of the coating shall be removed or repaired, prior to wash cleaning. Thermal cut edges shall be ground to reduce hardness to attain the surface profile required from abrasive blast cleaning.

All dirt, oil, and scaling shall be removed prior to blast cleaning. All surfaces shall be wash cleaned with either clean water at 8000 psi or water and detergent at 2000 psi with two rinses with clean water.

The surface shall be abrasive blast cleaned to near white metal (SSPC-SP 10). The surface profile shall be measured using a surface profile comparator, replica tape, or other method suitable for the abrasive being used in accordance with ASTM D4417.

Where zinc coatings up to and including 0.009 inch thick are to be applied, one of the following abrasive grits shall be used with pressure blast equipment to produce a 3.0 mils AA anchor tooth pattern:

1. Aluminum oxide or silicon carbide mesh size: SAE G-25 to SAE G-40
2. Hardened steel grit mesh size: SAE G-25 to SAE G-40
3. Garnet, flint, or crushed nickel or black beauty coal slag mesh size: SAE G-25 to SAE G-50

Where zinc coatings greater than 0.010 inch thick are to be applied, one of the following abrasive grits shall be used with pressure blast equipment to produce a 5.0 mils AA anchor tooth pattern:

1. Aluminum oxide or silicon carbide mesh size: SAE G-18 to SAE G-25
2. Hardened steel grit mesh size: SAE G-18 to SAE G-25
3. Garnet, flint, or crushed nickel or black beauty coal slag mesh size: SAE G-18 to SAE G-25

The pressure of the blast nozzle, as measured with a needle probe gauge, with pressure type blasting equipment shall be as follows:

1. With aluminum oxide, silicon carbide, flint, or slag - 50 psi minimum and 60 psi maximum.
2. With garnet or steel grit - 75 psi minimum.

The pressure at the blast nozzle, with siphon blasting (suction blasting), shall be as follows:

1. With aluminum oxide, silicon carbide, flint, or slag - 75 psi maximum.
2. With garnet or steel grit - 90 psi maximum.

The abrasive blast stream shall be directed onto the substrate surface at a spray angle of 75 to 90 degrees, and moved side to side. The nozzle to substrate distance shall be 4 to 12 inches.

6-07.3(14)F Application of Metallic Coating

Surfaces showing signs of condensed moisture or do not comply with Section 6-07.3(14)E shall not be sprayed. If rust bloom occurs within the holding time between abrasive blast cleaning and thermal spraying, the surface shall be reblasted at a blast angle as close to perpendicular to the surface as possible to achieve a 2.0 to 4.0 mil anchor tooth pattern. Thermal spraying shall not take place when the relative humidity is 90 percent or greater, when the steel temperature is less than 5F above the dew point, or when the air or steel temperature is less than 40F.

Clean, dry air shall be used with not less than 50 psi air pressure at the air regulator. Not more than 50 feet of ⅜ inch. ID hose shall be used between the air regulator and the metallizing gun. The metallizing gun shall be started and adjusted with the spray directed away from the work. During the spraying operation and depending upon the equipment being used, the gun shall be held as close to perpendicular as possible to the surface from 5 to 8 inches from the surface of the work.

Manual spraying shall be done in a block pattern, typically 2 feet by 2 feet square. The sprayed metal shall overlap on each pass to ensure uniform coverage. The specified thickness of the coating shall be applied in multiple layers. In no case are fewer than two passes of thermal spraying, overlapping at right angles, acceptable.

At least one single layer of coating shall be applied within 4 hours of blasting and the surface shall be completely coated to the specified thickness within 8 hours of blasting.

The minimum coating thickness shall be 6 mils unless otherwise shown in the Plans.

6-07.3(14)G Applications of Shop Coats and Field Coats

The surface shall be wiped clean with solvent immediately before applying the wash primer. The wash primer shall have a low viscosity appropriate for absorption into the thermal spray coating, and shall be applied within 8 hours after completion of thermal spraying or before oxidation occurs. The dry film thickness of the wash primer shall not exceed 0.5 mils or be less than 0.3 mils. It shall be applied using an appropriate spray gun except in those areas where brush or roller application is necessary. The subsequent shop primer coats shall be applied no less than one-half hour after a wash primer.

The shop primer coat, when specified, shall be applied in accordance with Section 6-07.3(11)A and the paint manufacturer's recommendations.

All field coats, when specified, shall be applied in accordance with Section 6-07.3(11)A and the paint manufacturer's recommendations. The color of the top coat shall conform to Section 6-03.3(30) as supplemented in these Special Provisions.

6-07.4 Measurement

Sealing and caulking pack rust will be measured by the linear foot along the edge of the steel connection interface sealed and caulked.

Spot abrasive blast cleaning of steel surfaces in accordance with Section 6-07.3(10)D will be measured by the square foot of surface area to be cleaned to bare metal as specified by the Engineer.

6-07.5 Payment

Payment will be made for each of the following Bid items that are included in the Proposal:

"Cleaning and Painting - _____", lump sum.

The lump sum Contract price for "Cleaning and Painting – _____" shall be full pay for the Work as specified, including developing all submittals; arranging for and accommodating contact and on-site attendance by the paint manufacturer's technical representative; furnishing and placing all necessary staging and rigging; furnishing, operating, and mooring barges; furnishing and operating fixed and movable work platforms; accommodating Contracting Agency inspection access; conducting the Contractor's quality control inspection program; providing material, labor, tools, and equipment; furnishing containers for containment waste, collecting and storing containment waste; collecting, storing, testing, and disposing of all containment waste not conforming to the definition in Section 6-07.3(10)F; performing all cleaning and preparation of surfaces to be painted, including removal of debris on pier caps supporting steel members and bearings included in the painting limits; cleaning and preparing areas needing treatment of pack rust and gaps; applying all coats of paint; correcting coating deficiencies; completing coating repairs; and completing project site cleanup.

When a weather station is specified, all costs in connection with furnishing, installing, operating, and removing the weather station, including furnishing mounting hardware and repeaters, accessories and wireless display console units, processing and submitting daily weather data reports, maintenance and upkeep, shall be included in the lump sum Contract price for "Cleaning And Painting – _____".

Progress payments for "Cleaning and Painting – _____" will be made on a monthly basis and will be based on the percentage of the total estimated area satisfactorily cleaned and coated as determined by the Engineer. Payment will not be made for areas that are otherwise complete but have repairs outstanding.

"Sealing and Caulking Pack Rust", per linear foot.

The unit contract price per linear foot for "Sealing and Caulking Pack Rust" shall be full pay for performing the work as specified, including applying the rust penetrating sealer, foam backer rod when applicable, and caulk.

"Spot Abrasive Blast Cleaning", per square foot.

The unit contract price per square foot for "Spot Abrasive Blast Cleaning" shall be full pay for performing the spot abrasive blast cleaning work in accordance with Section 6-07.3(10)D.

"Containment of Abrasives", lump sum.

The lump sum contract price for "Containment of Abrasives" shall be full payment for all costs incurred by the Contractor in complying with the requirements as specified in Section 6-07.3(10)A to design, construct, maintain, and remove containment systems for abrasive blasting operations.

"Testing and Disposal of Containment Waste", by force account as provided in Section 1-09.6.

All costs in connection with testing containment waste, transporting containment waste for disposal, and disposing of containment waste in accordance with Section 6-07.3(10)F will be paid by force account in accordance with Section 1-09.6. For the purpose of providing a common proposal for all bidders, the Contracting Agency has entered an amount for the item "Testing and Disposal of Containment Waste" in the bid proposal to become part of the total bid by the Contractor.

All costs in connection with producing the metallic coatings as specified shall be included in the unit contract price for the applicable item or items of work.

Payment for painting new steel structures and painting or powder coating of galvanized surfaces will be in accordance with Section 6-03.5. Painting of timber structures will be in accordance with Section 6-04.5.

6-08 Bituminous Surfacing on Structure Decks

6-08.1 Description

This Work consists of removing and placing Hot Mix Asphalt (HMA) or Bituminous Surface Treatment (BST) directly on or over a Structure. This Work also includes performing concrete bridge deck repair, applying waterproofing membrane, and sealing paving joints.

6-08.2 Materials

Materials shall meet the requirements of the following sections:

Bituminous Surface Treatment	5-02.2
Hot Mix Asphalt	5-04.2
Joint Sealants	9-04.2
Closed Cell Foam Backer Rod	9-04.2(3)A
Waterproofing Membrane (Deck Seal)	9-11
Bridge Deck Repair Material	9-20.5

6-08.3 Construction Requirements

6-08.3(1) Definitions

Adjusted Removal Depth – the Bituminous Pavement removal depth specified by the Engineer to supersede the Design Removal Depth after review of the Contractor survey of the existing Bituminous Pavement grade profile.

Bituminous Pavement – the surfacing material containing an asphalt binder.

Design Removal Depth – the value shown in the “pavement schedule” or elsewhere in the Plans to indicate the design thickness of Bituminous Pavement to be removed.

Final Grade Profile – the compacted finished grade surface of completed Bituminous Pavement surfacing consisting of a vertical profile and superelevation cross-slope, developed by the Engineer for Grade Controlled Structure Decks based on the Contractor survey.

Grade Controlled – a Structure Deck requiring restriction of Bituminous Pavement work, including restriction of pavement removal methods and restriction of overlay pavement thicknesses.

Structure Deck – the bridge deck (concrete or timber), bridge approach slab, top of concrete box Structure, or other concrete surfaces over or upon which existing Bituminous Pavement is removed and new Bituminous Pavement is applied.

6-08.3(2) Contractor Survey for Grade Controlled Structure Decks

Prior to removing existing Bituminous Pavement from a Grade Controlled Structure Deck, the Contractor shall complete a survey of the existing surface for use in establishing the existing cross section and grade profile elevations. When removal of Bituminous Pavement is to be achieved by rotary milling/planing, the Contractor’s survey shall also include the depths of the existing surfacing at each survey point.

The Contractor is responsible for all calculations, surveying, installation of control points, and measuring required for setting, maintaining and resetting equipment and materials necessary for the construction of the overlay to the Final Grade Profile.

6-08.3(2)A Survey Requirements

The Contractor shall establish at least two primary survey control points for controlling actual Bituminous Pavement removal depth and the Final Grade Profile. Horizontal control shall be by station and offset which shall be tied to either the Roadway centerline or the Structure centerline. Vertical control may be an assumed datum established by the Contractor.

Primary control points shall be described by station or milepost and offset on the baseline selected by the Contractor. The Contractor may expand the survey control information to include secondary horizontal and vertical control points as needed for the project.

Survey information collected shall include station or milepost, offset, and elevation for each lane line and curb line. Survey information shall be collected at even 20 foot station intervals, and along the centerline of each bridge expansion joint. The survey shall extend 300’-0” beyond the bridge back of pavement seat or end of Structure Deck. The survey information shall include the top of Bituminous Pavement elevation and, when rotary milling/planing equipment is used, the corresponding depth of Bituminous Pavement to the Structure Deck. The Contractor shall ensure a surveying accuracy to within ± 0.01 feet for vertical control and ± 0.2 feet for horizontal control.

Voids in HMA created by the Contractor’s Bituminous Pavement depth measurements shall be filled by material conforming to Section 9-20 or another material acceptable to the Engineer.

6-08.3(2)B Survey Submittal

The Contractor’s survey records shall include descriptions of all survey control points including station/milepost, offset, and elevations of all secondary control points. The Contractor shall maintain survey records of sufficient detail to allow the survey to be reproduced. The Contractor shall submit a Type 2 Working Drawing consisting of the compiled survey records and information. Survey data shall be submitted as an electronic file in Microsoft Excel format.

6-08.3(2)C Final Grade Profile and Adjusted Removal Depth

Based on the results of the survey, the Engineer may develop a Final Grade Profile and Adjusted Removal Depth. If they are developed, the Final Grade Profile and Adjusted Removal Depth will be provided to the Contractor within three working days after receiving the Contractor’s survey information. When provided, the Adjusted Removal Depth supersedes the Design Removal Depth to become the Bituminous Pavement removal depth for that Structure Deck.

6-08.3(3) General Bituminous Pavement Removal Requirements

Contractor shall remove Bituminous Pavement and associated deck repair material from Structure Decks to the horizontal limits shown in the Plans and to either the specified or adjusted Bituminous Pavement removal depth as applicable.

Removal of Bituminous Pavement within 12-inches of existing permanent features that limit the reach of the machine or the edge of the following items shall be by hand or by hand operated (nominal 30-pounds class) power tools: existing bridge expansion joint headers; steel expansion joint assemblies; concrete butt joints between back of pavement seats and bridge approach slabs, bridge drain assemblies; thrie beam post steel anchorage assemblies fastened to the side or top of the Structure Deck.

When removing Bituminous Pavement with a planer, Section 5-04.3(14) shall apply. If the planer contacts the Structure Deck in excess of the specified planing depth tolerance, or contacts steel reinforcing bars at any time, the Contractor shall immediately cease planing operations and notify the Engineer. Planing operations shall not resume until completion of the appropriate adjustments to the planing machine and receiving the Engineer’s concurrence to resume.

6-08.3(4) Partial Depth Removal of Bituminous Pavement from Structure Decks

The depth of surfacing removal, as measured to the bottom of the lowest milling groove generated by the rotary milling/planing machine shall be +0.01, -0.02-feet of the specified or Adjusted Removal Depth as applicable.

6-08.3(5) Full Depth Removal of Bituminous Pavement from Structure Decks

6-08.3(5)A Method of Removal

The Contractor shall perform full depth removal by a method that does not damage or remove the Structure Deck in excess of the specified Bituminous Pavement removal tolerance. The Contractor shall submit a Type 2 Working Drawing consisting of the proposed methods and equipment to be used for full depth removal.

6-08.3(5)B Planer Requirements for Full Depth Removal

The final planed surface shall have a finished surface with a tolerance of +0.01, -0.02 feet within the planed surface profile, as measured from a 10-foot straight edge. Multiple passes of planing to achieve smoothness will not be allowed.

In addition to Section 6-08.3(3), the planing equipment shall conform to the following additional requirements:

1. The cutting tooth spacing on the rotary milling head shall be less than or equal to ¼ inch.
2. The rotary milling/planing machine shall have cutting teeth that leave a uniform plane surface at all times. All teeth on the mill head shall be kept at a maximum differential tolerance of ⅜-inch between the shortest and longest tooth, as measured by a straight edge placed the full width of the rotary milling head.
3. Cutting tips shall be replaced when 30 percent of the total length of the cutting tip material remains.

Prior to each day's Bituminous Pavement removal operations, the Contractor shall confirm to the satisfaction of the Engineer that the rotary head cutting teeth are within the specified tolerances.

6-08.3(5)C Structure Deck Cleanup after Bituminous Pavement Removal

Waterproofing membrane that is loose or otherwise not firmly bonded to the Structure Deck shall be removed as an incidental component of the Work of surfacing removal. Existing waterproofing membrane bonded to the Structure Deck need not be removed.

6-08.3(6) Repair of Damage due to Bituminous Pavement Removal Operations

All concrete bridge deck, pavement seat, and steel reinforcing bar damage due to the Contractor's surfacing removal operations shall be repaired by the Contractor in accordance with Section 1-07.13, and as specified below.

Damaged concrete in excess of the specified Bituminous Pavement removal tolerance shall be repaired in accordance with Section 6-08.3(7), with the bridge deck repair material placed to the level of the surrounding bridge deck and parallel to the final grade paving profile.

Damaged steel reinforcing bar shall be repaired as follows:

1. Damage to steel reinforcing bar resulting in a section loss less than 20-percent of the bar with no damage to the surrounding concrete shall be left in place and shall be repaired by removing the concrete to a depth ¾-inches around the top steel reinforcing bar and placing bridge deck repair material accepted by the Engineer to the level of the bridge deck and parallel to the final grade paving profile.

2. Damage to steel reinforcing bar resulting in a section loss of 20-percent or more in one location, bars partially or completely removed from the bridge deck, or where there is a lack of bond to the concrete, shall be repaired by removing the adjacent concrete and splicing a new bar of the same size. Concrete shall be removed to provide a ¾-inch minimum clearance around the bars. The splice bars shall extend a minimum of 40 bar diameters beyond each end of the damage.

6-08.3(7) Concrete Deck Repair

This Work consists of repairing the concrete deck after Bituminous Pavement has been removed.

6-08.3(7)A Concrete Deck Preparation

The Contractor, with the Engineer, shall inspect the exposed concrete deck to establish the extent of bridge deck repair in accordance with Section 6-21.3(7). Areas of Structure Deck left with existing well bonded waterproof membrane after full depth Bituminous Pavement removal are exempt from this inspection requirement.

All loose and unsound concrete within the repair area shall be removed with jackhammers no heavier than the nominal 30 pound class or chipping hammers no heavier than the nominal 15 pound class, or other mechanical means acceptable to the Engineer, and operated at angles less than 45 degrees as measured from the surface of the deck to the tool. If unsound concrete exists around the existing steel reinforcing bars, or if the bond between concrete and steel reinforcing bar is broken, the Contractor shall remove the concrete to provide a ¾ inch minimum clearance to the bar. The Contractor shall take care to prevent damage to the existing steel reinforcing bars and concrete to remain.

After removing sufficient concrete to establish the limits of the repair area, the Contractor shall make ¾ inch deep vertical saw cuts and maintain square edges at the boundaries of the repair area. The exposed steel reinforcing bars and concrete in the repair area shall be abrasive blasted and blown clean just prior to placing the bridge deck repair material.

6-08.3(7)B Ultra-Low Viscosity, Two-Part Liquid, Polyurethane-Hybrid Polymer Concrete

The ultra-low viscosity, two-part liquid, polyurethane-hybrid polymer concrete shall be mixed in accordance with the manufacturer's recommendations.

Aggregate shall conform to the gradation limit requirements recommended by the manufacturer. The aggregate and the ultra-low viscosity, two-part liquid, polyurethane-hybrid polymer concrete shall be applied to the repair areas in accordance with the sequence and procedure recommended by the manufacturer.

All repairs shall be float finished flush with the surrounding surface within a tolerance of ⅛ inch of a straight edge placed across the full width and breadth of the repair area.

6-08.3(7)C Pre-Packaged Cement Based Repair Mortar

The Contractor shall mix the pre-packaged cement based repair mortar using equipment, materials and proportions, batch sizes, and process as recommended by the manufacturer.

All repairs shall be float finished flush with the surrounding surface within a tolerance of ⅛ inch of a straight edge placed across the full width and breadth of the repair area.

6-08.3(7)D Cure

All bridge deck repair areas shall be cured in accordance with the manufacturer's recommendations and attain a minimum compressive strength of 2,500 psi before allowing vehicular and foot traffic on the repair and placing waterproofing membrane on the bridge deck over the repair.

6-08.3(8) Waterproof Membrane for Structure Decks

This work consists of furnishing and placing a waterproof sheet membrane system over a prepared Structure Deck prior to placing an HMA overlay. The waterproof membrane system shall consist of a sheet membrane adhered to the Structure Deck with a primer.

The Contractor shall comply with all membrane manufacturer's installation recommendations.

6-08.3(8)A Structure Deck Preparation

The Structure Deck and ambient air temperatures shall be above 50°F and the Structure Deck shall be surface-dry at the time of the application of the primer and membrane.

All areas of a Structure Deck that have fresh cast bridge deck concrete less than 28 days old (not including bridge deck repair concrete placed in accordance with Section 6-08.3(7)) shall cure for a period of time recommended by the membrane manufacturer, or as specified by the Engineer, before application of the membrane.

The entire Structure Deck and the sides of the curb and expansion joint headers to the height of the HMA overlay shall be free of all foreign material such as dirt, grease, etc. Prior to applying the primer or sheet membrane, all dust and loose material shall be removed from the Structure Deck. All surface defects such as spalled areas, cracks, protrusions, holes, sharp edges, ridges, etc., and other surface imperfections greater than ¼ inch in width shall be corrected prior to application of the membrane.

6-08.3(8)B Applying Primer

The primer shall be applied to the cleaned deck surfaces at the rate according to the procedure recommended by the membrane manufacturer. All surfaces to be covered by the membrane shall be thoroughly and uniformly coated with primer. Structure Deck areas left with existing well bonded waterproof membrane after bituminous surfacing removal shall receive an application of primer in accordance with the membrane manufacturer's recommendations. Precautionary measures shall be taken to ensure that pools and thick layers of primer are not left on the deck surface. The membrane shall not be applied until the primer has cured or volatile material has substantially dissipated, in accordance with the membrane manufacturer's recommendations.

The primer and waterproof membrane shall extend from the bridge deck up onto the curb face and expansion joint header face the thickness of the HMA overlay. The membrane shall adhere to the vertical surface.

6-08.3(8)C Placing Waterproof Membrane

Membrane application shall begin at the low point on the deck, and continue in a lapped shingle pattern. The overlap shall be a minimum of six inches or greater if recommended by the membrane manufacturer. Membrane seams shall be sealed as recommended by the membrane manufacturer. Hand rollers or similar tools shall be used on the applied membrane to assure firm and uniform contact with the primed Structure surfaces.

The fabric shall be neatly cut and contoured at all expansion joints and drains. At expansion joints, the membrane shall be slack or folded to allow for Structure movements without stress to the membrane. The cuts at bridge drains shall be two right angle cuts made to the inside diameter of the bridge deck drain outlet, after which the corners of the waterproof membrane shall be turned down into the drains and laid in a coating of primer.

6-08.3(8)D Membrane Repair and Protection

The waterproof membrane will be visually inspected by the Engineer for uniformity, tears, punctures, bonding, bubbles, wrinkles, voids and other defects. All such deficiencies shall be repaired in accordance with the membrane manufacturer's recommendations prior to placement of the HMA overlay.

The membrane material shall be protected from damage due to the paving operations in accordance with the membrane manufacturer's recommendations. No traffic or equipment except that required for the actual waterproofing and paving operations will be permitted to travel or rest on the membrane until it is covered by the HMA overlay. The use of windrows is not allowed for laydown of HMA on a membrane.

Where waterproofing membrane is placed in stages or applied at different times, a strip of temporary paper shall be used to protect the membrane overlap from the HMA hand removal methods.

6-08.3(9) Placing Bituminous Pavement on Structure Decks

HMA overlay shall be applied on Grade Controlled Structure Decks using reference lines for vertical control in accordance with Section 5-04.3(3)C.

The compacted elevation of the HMA overlay on Structure Decks shall be within ± 0.02 feet of the specified overlay thickness or Final Grade Profile as applicable. Deviations from the final grade paving profile in excess of the specified tolerance and areas of non-conforming surface smoothness shall be corrected in accordance with Section 5-04.3(13).

Final grade Roadway transitions to a Structure Deck with Bituminous Pavement shall not exceed a 0.20 percent change in grade in accordance with the bridge deck transition for HMA overlay Standard Plan, unless shown otherwise in the Plans.

Final grade compacted HMA elevations shall be higher than an adjacent concrete edge by ¼ inch ± ⅛ inch at all expansion joint headers and concrete butt joints as shown in the concrete to asphalt butt joint details of the bridge paving joint seals Standard Plan. This also applies to steel edges within the limits of the overlay such as bridge drain frames and steel joint riser bars at bridge expansion joints.

6-08.3(9)A Protection of Structure Attachments and Embedments

The Contractor is responsible for protecting all Structure attachments and embedments from the application of BST and HMA.

Drainage inlets that are to remain open, and expansion joints, shall be cleaned out immediately after paving is completed. Materials passing through expansion joints shall be removed from the bridge within 10 working days.

All costs incurred by the Contractor in protective measures and clean up shall be included in the unit Contract prices for the associated Bid items of Work.

6-08.3(10) HMA Compaction on Structure Decks

Compaction of HMA on Structure Decks shall be in accordance with Section 5-04.3(10).

Work rejected in accordance with Section 5-04.3(11) shall include the materials, work, and incidentals to repair an existing waterproof membrane damaged by the removal of the rejected work.

6-08.3(11) Paved Panel Joint Seals and HMA Saw Cut and Seals

Unless otherwise required, bridge paving joint seals shall be installed in accordance with Section 5-03.3(4) or 5-03.3(5) as appropriate.

6-08.4 Measurement

Removing existing Bituminous Pavement from Structure Decks will be measured by the square yard of Structure Deck surface area with removed overlay.

Bridge deck repair will be measured by the square foot surface area of deck concrete removed with the measurement taken at the plane of the top mat of steel reinforcing bars.

Waterproof membrane will be measured by the square yard surface area of Structure Deck and curb and header surface area covered by membrane.

6-08.5 Payment

Payment will be made for each of the following Bid items that are included in the Proposal:

> "Structure Surveying", lump sum.
>
> "Removing Existing Overlay From Bridge Deck___", per square yard.
>
> The unit Contract price per square yard for "Removing Existing Overlay From Bridge Deck___", shall be full pay for performing the Work as specified for full removal of Bituminous Pavement on Structure Decks, including the removal of existing waterproof membrane and disposing of materials.
>
> "Bridge Deck Repair Br. No.___", per square foot.
>
> The unit Contract price per square foot for "Bridge Deck Repair Br. No.___" shall be full pay for performing the Work as specified, including removing and disposing of the concrete within the repair area and furnishing, placing, finishing, and curing the repair concrete.
>
> "Waterproof Membrane Br. No.___", per square yard.
>
> The unit Contract price per square yard for "Waterproof Membrane Br. No.___" shall be full pay for performing the Work as specified, including repairing any damaged or defective waterproofing membrane and repair of damaged HMA overlay.

6-09 Vacant

6-10 Concrete Barrier

6-10.1 Description

This section applies to building precast or cast-in-place cement concrete barriers as required by the Plans, these Specifications, or the Engineer.

This Work may also include the removal, storage and resetting of permanent barrier at the locations shown in the Plans or as specified by the Engineer.

6-10.2 Materials

Materials shall meet the requirements of the following sections:

Cement	9-01
Aggregates	9-03
Premolded Joint Fillers	9-04.1
Reinforcing Steel	9-07
Grout	9-20.3

Wire rope shall be Class 6 × 19, made of improved plow steel that has been galvanized and preformed. Galvanizing shall meet ASTM A603. The wire rope shall have right regular lay and a fiber core. It shall be ⅝ inch in diameter and have a minimum breaking strength of 15 tons.

All hardware (connecting pins, drift pins, nuts, washers, etc.) shall be galvanized in accordance with AASHTO M 232.

Connecting pins, drift pins and steel pins for type 3 anchors shall conform to Section 9-06.5(4) and be galvanized in accordance with AASHTO M232. All other hardware shall conform to Section 9-06.5(1) and be galvanized in accordance with AASHTO M232.

Grout for permanent installations of precast single slope barrier shall conform to Section 9-20.3(3) and shall be placed in accordance with Section 6-02.3(20).

6-10.3 Construction Requirements

Single slope barrier shall be cast-in-place or slipformed, except when precast single slope barrier is specified in the Plans or specified by the Engineer. Concrete barrier installed in conjunction with light standard foundations regardless of the barrier shape, shall be cast-in-place using stationary forms.

Steel welded wire reinforcement deformed, conforming to Section 9-07.7, may be substituted in concrete barrier in place of deformed steel bars conforming to Section 9-07.2, subject to the following conditions:

1. Steel welded wire reinforcement spacing shall be the same as the deformed steel bar spacing shown in the *Standard Plans*.
2. The minimum cross sectional area for steel welded wire reinforcement shall be no less than 86 percent of the cross sectional area for the deformed steel bars being substituted.
3. Development lengths and splice lengths shall conform to requirements specified in the AASHTO *LRFD Bridge Design Specifications*, current edition.

6-10.3(1) Precast Concrete Barrier

Precast concrete barrier shall meet all the requirements in Section 6-02.3(9), except as modified in this section.

Test results from the QC testing shall demonstrate compliance with Sections 6-02.3(4)C consistency, 6-02.3(4)D temperature and time of placement, 6-02.3(2)A air content, and compressive strength. All tests will be conducted in accordance with Section 6-02.3(5)D.

The QC tester conducting the sampling and testing shall be qualified by ACI, Grade I to perform this Work. The equipment used shall be calibrated/certified annually.

All test results and certifications shall be kept at the fabricator's facility for review by the Contracting Agency.

The Contracting Agency intends to perform Quality Assurance Inspection. This inspection is for the qualification of the plant QC process. This inspection shall not relieve the Contractor of any responsibility for identifying and replacing defective material and Work.

The concrete in precast barrier shall be Class 5000 for Type F and Class 4000 for all other precast barriers, and comply with the provisions of Section 6-02.3. No concrete barrier shall be shipped until test cylinders made of the same concrete and cured under the same conditions show the concrete has reached the specified 28 day compressive strength.

The Contractor may use Type III portland cement, but shall bear any added cost.

Precast barrier shall be cast in steel forms. After release, the barrier shall be finished to an even, smooth, dense surface, free from any rock pockets or holes larger than ¼ inch across. Troweling shall remove all projecting concrete from the bearing surface.

If heat curing methods are used, precast concrete barrier shall be cured in accordance with Section 6-02.3(25)D except that the barrier shall be cured in the forms until a rebound number test, or test cylinders which have been cured under the same conditions as the barrier, indicate the concrete has reached a compressive strength of a least 70% of the specified 28 day compressive strength. No additional curing is required once the barrier is removed from the forms.

The barrier shall be precast in sections as the *Standard Plans* require. All barrier in the same project (except end sections and variable length units needed for closure) shall be the same length. All barrier shall be new and unused. It shall be true to Plan dimensions. The manufacturer shall be responsible for any damage or distortion that results from manufacturing.

Only one section less than 20 feet long for single slope barrier and 10 feet long for all other barriers may be used in any single run of precast barrier, and it shall be at least 8 feet long. It may be precast or cast-in-place. Hardware identical to that used with other sections shall interlock such a section with adjacent precast sections.

Barrier connection voids for permanent installations of precast single slope barrier shall be filled with grout.

6-10.3(2) Cast-In-Place Concrete Barrier

Forms for cast-in-place concrete barrier, including traffic barrier, traffic-pedestrian barrier, and pedestrian barrier on bridges and related Structures, shall be made of steel or exterior plywood coated with plastic. The Contractor may construct the barrier by the slip-form method.

The barrier shall be made of Class 4000 concrete that meets the requirements of Section 6-02, except that the fine aggregate gradation used for slip-form barrier may be either Class 1 or 2. The Contractor may use portland cement Type III at no additional expense to the Contracting Agency.

In addition to the steel reinforcing bar tying and bracing requirements specified in Section 6-02.3(24) C, the Contractor may also place small amounts of concrete to aid in holding the steel reinforcing bars in place. These small amounts of concrete shall be not more than 2-cubic feet in volume, and shall be spaced at a minimum of 10-foot intervals within the steel reinforcement cage. The Contractor may request an alternate size and/or spacing for concrete used to anchor reinforcing by submitting a Type 2 Working Drawing. These small amounts of concrete shall be consolidated in accordance with Section 6-02.3(6) and shall provide 2 inches minimum clearance to the steel reinforcing bars on the outside face of the barrier. All spattered and excess mortar and concrete shall be removed from the steel reinforcing bars prior to slip-form casting.

Barrier expansion joints shall be spaced at 96-foot intervals, and dummy joints shall be spaced at 12-foot intervals unless otherwise specified in the Contract.

Immediately after removing the forms, the Contractor shall complete any finishing Work needed to produce a uniformly smooth, dense surface. The surface shall have no rock pockets and no holes larger than ¼ inch across. The barrier shall be cured and finished in accordance with Section 6-02.3(11)A.

The maximum allowable deviation from a 10-foot straightedge held longitudinally on all surfaces shall be ¼ inch. For single sloped barrier the maximum allowable deviation from a straightedge held along the vertical sloped face of the barrier shall be ¼ inch.

At final acceptance of the project, the barrier shall be free from stains, smears, and any discoloration.

6-10.3(2)A Moment Slab Traffic Barrier

Moment slab traffic barrier 42” single slope shall be constructed as detailed in the Plans. Finishing and curing shall be in accordance with Section 6-02.3(11)A. The moment slab supporting the barrier shall be continuously wet cured for 3 days in accordance with Section 6-02.3(11).

6-10.3(2)A1 Moment Slab Traffic Barrier and SEW Pedestrian Barrier on a Structural Earth Wall

When Moment slab traffic barrier 42” single slope is to be constructed on a structural earth wall, the Contractor shall coordinate with the structural earth wall manufacturer so that the structural earth wall manufacturer can design the structural earth wall and provide shop drawings for the wall that do not conflict with the moment slab traffic barrier.

When SEW Pedestrian Barrier is to be installed on a structural earth wall, the SEW Pedestrian Barrier shall be contractor designed and have an integral moment slab. The Contractor, in conjunction with the structural earth wall manufacturer, shall design and detail the SEW Pedestrian Barrier in accordance with Section 6-13.3(2) and the above ground geometry details shown in the Plans. The Contractor shall submit Type 2E working drawings and supporting calculations for the SEW Pedestrian Barrier. The working drawings shall include, at a minimum, the following:

1. Complete details of barrier cross section geometry, including the portion below ground, and accommodations necessary for bridge approach slabs, PCCP, drainage facilities, underground utilities, and sign support, luminaire pole, traffic signal standard, and other barrier attachments.
2. Details of the steel reinforcement of the barrier, including a bar list and bending diagram in accordance with Section 6-02.3(24), and including additional reinforcement required at sign support, luminaire pole, traffic signal standard, and other barrier attachment locations.
3. Details of the interface of, and the interaction between, the barrier and the top layers of structural earth wall reinforcement and facing.

4. When the Plans specify placement of conduit pipes through the barrier, details of conduit pipe and junction box placement.

SEW pedestrian barrier shall be constructed in accordance with Sections 6-02.3(11)A and 6-10.3(2), and the details in the Plans and in the SEW pedestrian barrier Working Drawings as approved by the Engineer. The moment slab supporting the SEW pedestrian barrier shall be continuously wet cured for 3 days in accordance with Section 6-02.3(11).

6-10.3(3) Removing and Resetting Permanent Concrete Barrier

The Contractor shall reset concrete barrier if the Plans or the Engineer require. If resetting is impossible immediately after removal, the Contractor shall store the barrier at Engineer-approved locations.

6-10.3(4) Joining Precast Concrete Barrier to Cast-In-Place Barrier

The Contractor may join segments of cast-in-place barrier to precast barrier where transitions, split barriers, or gaps shorter than 10 feet require it. At each joint of this type, the cast-in-place segment shall include hardware that ties both its ends to abutting precast sections.

6-10.3(5) Temporary Barrier

For temporary barrier, the Contractor may use Type F precast concrete barrier, Type 2 precast concrete barrier fabricated on or before December 31, 2019, or temporary steel barrier. If temporary steel barrier is selected, the Contractor shall verify the lateral deflection distance meets or is less than what is shown in the Contract Plans. Temporary concrete barrier shall comply with Standard Plan requirements and cross-sectional dimensions, except that: (1) it may be made in other lengths than those shown in the Standard Plan, and (2) it may have permanent lifting holes no larger than 4-inches in diameter or lifting loops. New Type 2 and Type 4 precast barrier shall not be fabricated for use on the project. Temporary steel barrier shall be certified that it meets the requirements of NCHRP 350 or MASH Test Level 3 or 4 as specified in Section 2-04.3(3). Temporary steel barrier shall be installed in accordance with the manufacturer’s recommendations.

New Type 2 and Type 4 precast barrier shall not be fabricated.

If the Contract calls for the removal and resetting of permanent barrier, and the permanent barrier is not required to remain in place until reset, the permanent barrier may be substituted for temporary concrete barrier. Any of the permanent barrier damaged during its use as temporary barrier will become the property of the Contractor and be replaced with permanent barrier when the permanent barrier is reset to its permanent location.

All barrier shall be in good condition, without cracks, chips, spalls, dirt, or traffic marks. If any barrier segment is damaged during or after placement, the Contractor shall immediately repair it to the Engineer’s satisfaction or replace it with an undamaged section.

Delineators shall be placed on the traffic face of the barrier 6 inches from the top and spaced a maximum of 40 feet on tangents and 20 feet through curves. The reflector color shall be white on the right side of traffic and yellow on the left side of traffic. The Contractor shall maintain, replace and clean the delineators when ordered by the Engineer.

As soon as the temporary barrier is no longer needed, the Contractor shall remove it from the project. Contracting Agency furnished barrier shall remain Contracting Agency property, and the Contractor shall deliver it to a stockpile site noted in the Contract or to locations as approved by the Engineer. Contractor furnished barrier shall remain the property of the Contractor.

6-10.3(6) Placing Concrete Barrier

Permanent precast concrete barrier Type F, Types 2 and 4, and transitions shall rest on a paved foundation shaped to a uniform grade and section. Temporary precast Type F, Types 2 and 4 and transitions shall rest on a paved foundation or compacted unpaved surface shaped to a uniform grade and section. The foundation surface for permanent and temporary precast concrete barrier Type F, Types 2 and 4, and transitions shall meet this test for uniformity: When a 10-foot straightedge is placed on the surface parallel to the centerline for the barrier, the surface shall not vary more than ¼ inch from the lower edge of the straightedge. If deviations exceed ¼ inch, the Contractor shall correct them as required in Section 5-04.3(13).

The Contractor shall align the joints of all precast segments so that they offset no more than ¼ inch transversely and no more than ¾ inch vertically. Grouting is not permitted. If foundation grade and section are acceptable, the Engineer may permit the Contractor to obtain vertical alignment of the barrier by shimming. Shimming shall be done with a polystyrene, foam pad (12 by 24 inches) under the end 12 inches of bearing surface.

Precast barrier (Single Slope, Type F, Types 2 and 4, and transitions) shall be handled and placed with equipment that will not damage or disfigure it.

6-10.4 Measurement

Precast concrete barrier will be measured by the linear foot along its completed line and slope.

Temporary barrier will be measured by the linear foot along the completed line and slope of the barrier, one time only for each setup of barrier protected area. Any intermediate moving or resetting will not be measured.

Cast-in-place concrete barrier, moment slab traffic barrier, and SEW pedestrian barrier, will be measured by the linear foot along its completed line unless the Contract specifies that it be measured per cubic yard for concrete Class 4000 and per pound for steel reinforcing bar (as required in Section 6-02.4).

Cast-in-place concrete barrier light standard section will be measured by the unit for each light standard section installed.

Removing and resetting existing permanent barrier will be measured by the linear foot and will be measured one time only for removing, storage, and resetting. No measure will be made for barrier that has been removed and reset for the convenience of the Contractor.

Concrete barrier transition Type 2 to bridge F-shape will be measured by the linear foot installed.

Single slope concrete barrier light standard foundation will be measured by the unit for each light standard foundation installed.

Traffic barrier, traffic pedestrian barrier, and pedestrian barrier will be measured as specified for cast-in-place concrete barrier.

6-10.5 Payment

Payment will be made for each of the following Bid items that are included in the Proposal:

“Precast Conc. Barrier Type ____”, per linear foot.

“Precast Conc. Barrier Type F Unanchored”, per linear foot.

“Precast Conc. Barrier Type F Anchored”, per linear foot.

“Cast-In-Place Conc. Barrier”, per linear foot.

“Conc. Class 4000 _____”, per cubic yard.

"St. Reinf. Bar _____", per pound.

"Removing and Resetting Existing Permanent Barrier", per linear foot.

The unit Contract price per linear foot for "Cast-In-Place Conc. Barrier" shall be full pay for excavation, forms, placement, special construction features, and all other materials, tools, equipment, and labor necessary to complete the Work as specified; except that when the Contract specifies, the unit Contract price per cubic yard for "Conc. Class 4000 _____" and the per pound for "St. Reinf. Bar _____", shall be full pay for excavation, forms, placement, special construction features, and all other materials, tools, equipment, and labor necessary to complete the Work as specified.

"Moment Slab Traffic Barrier", per linear foot.

The unit Contract price per linear foot for "Moment Slab Traffic Barrier" shall be full pay for for all work necessary to construct the moment slab traffic barrier including the associated moment slab as specified.

"SEW Pedestrian Barrier", per linear foot

The unit Contract price per linear foot for "SEW Pedestrian Barrier" shall be full pay for all work necessary to construct the SEW pedestrian barrier including the associated moment slab as specified.

"Traffic Barrier", per linear foot.

"Traffic Pedestrian Barrier", per linear foot.

"Pedestrian Barrier" per linear foot.

The unit Contract price per linear foot for "Traffic Barrier", "Traffic Pedestrian Barrier", and "Pedestrian Barrier" shall be full pay for constructing the barrier on top of the bridge deck, and associated bridge approach slabs, curtain walls and wingwalls, excluding the steel reinforcing bars that extend from the bridge deck, bridge approach slab, curtain walls, and wingwalls.

"Single Slope Concrete Barrier", per linear foot.

The unit Contract price per linear foot for "Single Slope Concrete Barrier" shall be full pay for either cast-in-place or precast single slope concrete barrier.

"Conc. Barrier Transition Type 2 to Bridge F-Shape", per linear foot.

The unit Contract price per linear foot for "Conc. Barrier Transition Type 2 to Bridge F-Shape" shall be full pay for performing the Work as specified, excluding bridge traffic barrier modifications necessary for this installation.

"Single Slope Conc. Barrier Light Standard Foundation", per each.

"Cast-In-Place Conc. Barrier Light Standard Section", per each.

"Temporary Barrier", per linear foot.

The unit Contract price per linear foot for "Temporary Barrier" shall be full pay for all costs, including furnishing, installing, connecting, anchoring, maintaining, temporary storage, and final removal of the temporary barrier.

Payment for transition sections between different types of barrier shall be made at the unit Contract price for the type of barrier indicated in the Plans for each transition section.

6-11 Reinforced Concrete Walls

6-11.1 Description

This Work consists of constructing reinforced concrete retaining walls, including those shown in the *Standard Plans*.

6-11.2 Materials

Materials shall meet the requirements of the following sections:

Cement	9-01
Aggregates for Concrete	9-03.1
Gravel Backfill	9-03.12
Premolded Joint Filler	9-04.1(2)
Sealing Band	9-04.12
Steel Reinforcing Bar	9-07.2
Epoxy-Coated Steel Reinforcing Bar	9-07.3
Welded Wire Reinforcement	9-07.7
Concrete Surface Treatments	9-08.3
Grout	9-20.3(2)
Concrete Curing Materials and Admixtures	9-23
Fly Ash	9-23.9
Water	9-25

6-11.3 Construction Requirements

6-11.3(1) Submittals

All components of reinforced concrete retaining walls, regardless of the combination of precast and cast-in-place components shall be submitted simultaneously as a comprehensive submittal.

6-11.3(1)A Precast Reinforced Concrete Retaining Walls

When a precast reinforced concrete retaining wall using Standard Plan D-20.10 is detailed in the Plans, the Contractor shall submit a Type 2 Working Drawing of the precast unit shop drawings in accordance with Section 6-02.3(9)A.

When cast-in-place footing keys are required, the precast unit shop drawing shall also include the following:

1. The construction method option selected from the Plans
2. The anticipated trench excavation wall slopes
3. The methods for dewatering if required
4. The methods for maintaining stability of the walls prior to and during placement of the footing key concrete
5. The location and size of block outs and closure holes.

6-11.3(1)B Cast-In-Place Reinforced Conc. Retaining Walls

When cast-in-place reinforced concrete retaining walls are called out in the Plans, the Contractor shall submit Type 2E Working Drawings of falsework and formwork plans in accordance with Sections 6-02.3(16) and 6-02.3(17).

6-11.3(1)B1 Substitution of Precast Stem Walls in Lieu of Cast-In-Place Stem Walls

The Contractor may elect to fabricate and erect precast reinforced concrete wall stem panels in place of the cast-in-place wall stem panels.

If the Contractor elects to use precast wall stem panels in lieu of cast-in-place wall stem panels, Type 2E Working Drawings shall be submitted that meet the requirements of Section 6-11.3(1)A and also include the following:

1. Working drawings for fabrication of the precast wall stem panels, showing dimensions, steel reinforcing bars, joint and joint filler details, surface finish details, lifting devices with the manufacturer's recommended safe working capacity, and material Specifications.
2. Working drawings and design calculations for the erection of the precast wall stem panels showing dimensions, support points, support footing sizes, erection blockouts, member sizes, connections, and material Specifications.
3. Design calculations for the precast wall stem panels, the connection between the precast panels and the cast-in-place footing, and all modifications to the cast-in-place footing details as shown in the Plans.
4. Cast-in-place submittal requirements for foundations in accordance with Section 6-11.3(1)A.

6-11.3(2) Excavation and Foundation Preparation

Excavation shall conform to Section 3-07.3(3), and to the limits and construction stages shown in the Plans. Foundation soils found to be unsuitable shall be removed and replaced in accordance with Section 3-07.3(1)C.

Bedding material for precast reinforced concrete retaining wall units shall be in accordance with the Standard Plans and Section 6-20.3(6)A.

6-11.3(3) Wall Construction

6-11.3(3)A Precast Reinforced Concrete Wall Construction

Precast reinforced concrete retaining wall units for Standard Plan D 20-10 and precast reinforced concrete wall stem panels shall conform to Section 6-02.3(9) except as modified in this section.

When precast reinforced concrete retaining walls are called out in the Plans to be constructed in accordance with Standard Plan D 20.10, the units shall be Class 7000 concrete. Cast-in-place footing keys shall be Class 4000 when required. The precast units shall be fabricated full height, and shall be fabricated in segment lengths greater than or equal to 4 feet.

When the Contractor elects to use precast stem panels as described in Section 6-11.3(1)B1, precast reinforced concrete stem panels shall be Class 4000 concrete unless otherwise shown in the Plans. The precast wall stem panels shall be fabricated full height and shall be fabricated in lengths of 8, 16, or 24 feet.

6-11.3(3)A1 Fabrication Tolerances

The construction tolerances for the precast reinforced concrete retaining wall units for Standard Plan D 20-10 and the precast reinforced concrete wall stem panels shall be as follows:

Height	±¼ inch
Width	±¼ inch
Thickness	+¼ inch
	-⅛ inch
Concrete cover for steel reinforcing bar	+⅜ inch
	-⅛ inch
Width of precast concrete wall stem panel joints	±¼ inch
Offset of precast concrete wall stem panels	±¼ inch

(Deviation from a straight line extending 5 feet on each side of the panel joint)

When precast reinforced concrete retaining walls are called out in the Plans to be constructed in accordance with Standard Plan D 20.10, the precast reinforced concrete retaining wall shall be constructed with a joint between adjacent units. The wall and footing joints shall be constructed as shown in the Standard Plans. The joints shall be continuous and shall be of uniform width over the entire height of the precast wall and footing.

When the Contractor elects to use precast stem panels as described in Section 6-11.3(1)B1, the precast concrete wall stem panels shall be constructed with a mating shear key between adjacent panels. The shear key shall have beveled corners and shall be 1½ inches in thickness. The width of the shear key shall be 3½ inches minimum and 5½ inches maximum. The shear key shall be continuous and shall be of uniform width over the entire height of the precast reinforced wall stem panel.

6-11.3(3)A2 Finishing

For precast reinforced concrete retaining wall units for Standard Plan D 20-10 and precast reinforced concrete wall stem panels, the Contractor shall provide the specified exterior concrete surface finish as noted, and to the limits shown, in the Plans. Surface finishes shall conform to Section 6-02.3(14) as supplemented in the Special Provisions. Rolled on textured finished shall not be used. If the Plans call for a form liner texture on both sides of the wall, it shall be cast in a vertical position.

6-11.3(3)A3 Erection

When precast reinforced concrete retaining walls are called out in the Plans to be constructed in accordance with Standard Plan D 20.10, all joints shall be constructed with sealing band installed on the rear (backfill) side of the precast reinforced concrete retaining walls. When cast-in-place footing keys are required, the precast reinforced concrete retaining walls shall be secured in place during placement and curing of the Class 4000 cast-in-place footing key. The Contractor shall ensure the concrete is fully consolidated around all headed reinforcing bars that are wet inserted into the Class 4000 concrete.

When the Contractor elects to use precast stem panels as described in Section 6-11.3(1)B1, the precast reinforced concrete wall stem panel shall be rigidly held in place during placement and curing of the cast-in-place footing concrete. The precast reinforced concrete wall stem panels shall be placed a minimum of 1 inch into the cast-in-place footing to provide a shear key. The base of the precast reinforced concrete wall stem panel shall be sloped ½ inch per foot to facilitate proper concrete placement. To ensure an even flow of concrete under and against the base of the precast reinforced concrete wall stem panel, a form shall be placed parallel to the precast reinforced concrete wall stem panel, above the cast-in-place footing, to allow a minimum 1-foot head to develop in the concrete during concrete placement. The steel reinforcing bars shall be shifted to clear the erection blockouts in the precast reinforced concrete wall stem panel by 1½ inches minimum. All joints shall be constructed with joint filler installed on the rear (backfill) side of the wall. The joint filler material shall extend from 2 feet below the final ground level in front of the wall to the top of the wall. The joint filler shall be a nonorganic flexible material and shall be installed to create a waterproof seal at panel joints. The soil bearing pressure beneath the falsework supports for the precast reinforced concrete wall stem panels shall not exceed the maximum design soil pressure shown in the Plans for the reinforced concrete retaining wall.

6-11.3(3)B Cast-In-Place Concrete Construction

Cast-in-place concrete for reinforced concrete retaining walls shall be formed, reinforced, cast, cured, and finished in accordance with Section 6-02, and the details shown in the Plans. All cast-in-place concrete shall be Class 4000 unless otherwise shown in the Plans. Cast-in-place footings shall have a longitudinal slope no steeper than 1V: 6H, unless otherwise shown in the Plans.

The Contractor shall provide the specified exterior concrete surface finish as noted, and to the limits shown in the Plans. Surface finishes shall conform to Section 6-02.3(14).

Cast-in-place concrete for adjacent wall stem sections (between vertical expansion joints) shall be formed and placed separately, with a minimum 24-hour time period between concrete placement operations.

Premolded joint filler, ½ inch thick, shall be placed full height of all vertical wall stem expansion joints in accordance with Section 6-01.14.

6-11.3(4) Backfill, Weepholes, and Gutters

Unless the Plans specify otherwise, backfill and weepholes shall be placed in accordance with the Plans and Section 6-02.3(22). Gravel backfill for drain shall be compacted in accordance with Section 3-07.3(1)E. Backfill within the zone defined as Bridge Approach Embankment in Section 1-01.3 shall be compacted in accordance with Method C of Section 3-03.3(14)C. All other backfill shall be compacted in accordance with Method B of Section 3-03.3(14)C, unless otherwise specified.

Cement concrete gutter shall be constructed as shown in the Plans or *Standard Plans*.

6-11.3(5) Traffic Barrier and Pedestrian Barrier

When shown in the Plans, traffic barrier and pedestrian barrier shall be constructed in accordance with Sections 6-02.3(11)A and 6-10.3(2), and the details shown in the Plans and *Standard Plans*.

6-11.4 Measurement

Concrete Class 4000 for retaining wall will be measured as specified in Section 6-02.4.

Except as noted below, concrete Class 7000 for precast retaining wall will be measured as specified in Section 6-02.4.

Except as noted below, all reinforcing steel for retaining wall and precast retaining wall will be measured as specified in Section 6-02.4.

Exception: When precast retaining walls are called out in the Plans to be constructed in accordance with Standard Plan D 20.10 with footing keys, the construction of the footing keys shall be incidental to wall construction. The concrete and reinforcing steel, including dowels, for the construction of footing keys will not be measured.

Traffic barrier and pedestrian barrier will be measured as specified in Section 6-10.4 for cast-in-place concrete barrier.

6-11.5 Payment

Payment will be made for each of the following Bid items when they are included in the Proposal:

“Conc. Class 4000 For Retaining Wall”, per cubic yard.

All costs in connection with furnishing and installing PVC pipe for weep holes, premolded joint filler, grout, exterior surface finish, and pigmented sealer (when specified), shall be included in the unit Contract price per cubic yard for “Conc. Class 4000 For Retaining Wall”.

“Conc. Class 7000 For Precast Retaining Wall”, per cubic yard.

All costs in connection with furnishing and installing PVC pipe for weep holes, premolded joint filler, joint sealant, external sealing bands, weld tie assemblies, footing keys, wall joints, footing joints, grout, exterior surface finish, and pigmented sealer (when specified), shall be included in the unit Contract price per cubic yard for “Conc. For Retaining Wall”.

“St. Reinf. Bar For Retaining Wall”, per pound.

“Epoxy-Coated St. Reinf. Bar For Retaining Wall”, per pound.

“St. Reinf. Bar For Precast Retaining Wall”, per pound.

“Epoxy-Coated St. Reinf. Bar For Precast Retaining Wall”, per pound.

Structure Excavation Class A and Shoring or Extra Excavation Class A will be paid for in accordance with Section 3-07.5.

Traffic and Pedestrian Barrier shall be paid for in accordance with Section 6-10.5.

6-12 Noise Barrier Walls

6-12.1 Description

This Work consists of constructing cast-in-place concrete, precast concrete, masonry, and timber noise barrier walls, including those shown in the *Standard Plans*.

6-12.2 Materials

Materials shall meet the requirements of the following sections:

Cement	9-01
Aggregates for Concrete	9-03.1
Gravel Backfill	9-03.12
Premolded Joint Filler	9-04.1(2)
Bolts, Nuts, and Washers	9-06.5(1)
Noise Barrier Wall Access Door	9-06.17
Steel Reinforcing Bar	9-07.2
Epoxy-Coated Steel Reinforcing Bar	9-07.3
Paints	9-08
Grout	9-20.3
Concrete Curing Materials and Admixtures	9-23
Fly Ash	9-23.9
Water	9-25

Other materials required shall be as specified in the Special Provisions.

6-12.3 Construction Requirements

6-12.3(1) Submittals

All noise barrier walls not constructed immediately adjacent to the Roadway, and that require construction of access for Work activities, shall have a noise barrier wall access plan. The Contractor shall submit a Type 2 Working Drawing consisting of the noise barrier wall access plan. The noise barrier wall access plan shall include, but not be limited to, the locations of access to the noise barrier wall construction sites, and the method, materials, and equipment used to construct the access, remove the access, and recontour and reseed the disturbed ground.

For construction of all noise barrier walls with shafts, the Contractor shall submit a Type 2 Working Drawing consisting of the shaft construction plan, including at a minimum the following information:

1. List and description of equipment to be used to excavate and construct the shafts, including description of how the equipment is appropriate for use in the expected subsurface conditions.
2. The construction sequence and order of shaft construction.
3. Details of shaft excavation methods, including methods to clean the shaft excavation.
4. Details and dimensions of the shaft, and casing if used.
5. The method used to prevent ground caving (temporary casing, slurry, or other means).
6. Details of concrete placement including procedures for deposit through a conduit, tremie, or pump.
7. Method and equipment used to install and support the steel reinforcing bar cage.

For construction of precast concrete noise barrier walls, the Contractor shall submit Type 2 Working Drawings consisting of shop drawings for the precast concrete panels in accordance with Section 6-02.3(9)A. In addition to the items listed in Section 6-02.3(9)A, the precast concrete panel shop drawings shall include the following:

1. Construction sequence and method of forming the panels.
2. Details of additional reinforcement provided at lifting and support locations.
3. Method and equipment used to support the panels during storage, transporting, and erection.
4. Erection sequence, including the method of lifting the panels, placing and adjusting the panels to proper alignment and grade, and supporting the panels during bolting, grouting, and backfilling operations.

The Contractor shall not begin noise barrier wall construction activities, including access construction and precast concrete panel fabrication, until receiving the Engineer's approval of all appropriate and applicable submittals.

6-12.3(2) Work Access and Site Preparation

The Contractor shall construct Work access in accordance with the Work access plan. The construction access roads shall minimize disturbance to the existing vegetation, especially trees. Only trees and shrubs in direct conflict with the approved construction access road alignment shall be removed. Only one access road into the noise barrier wall from the main Roadway and one access road from the noise barrier wall to the main Roadway shall be constructed at each noise barrier wall.

Existing vegetation that has been identified by the Engineer shall be protected in accordance with Sections 1-07.16 and 3-01, and the Special Provisions.

6-12.3(3) Shaft Construction

The Contractor shall excavate and construct the shafts in accordance with the shaft construction plan.

The shafts shall be excavated to the required depth as shown in the Plans. The excavation shall be completed in a continuous operation using equipment capable of excavating through the type of material expected to be encountered.

If the shaft excavation is stopped, the Contractor shall secure the shaft by installing a safety cover over the opening. The Contractor shall ensure the safety of the shaft and surrounding soil and the stability of the side walls. A temporary casing, slurry, or other methods acceptable to the Engineer shall be used as necessary to ensure such safety and stability.

When caving conditions are encountered, the Contractor shall stop further excavation until implementing the method to prevent ground caving as specified in the shaft construction plan.

When obstructions are encountered, the Contractor shall notify the Engineer promptly. An obstruction is defined as a specific object (including, but not limited to, boulders, logs, and man made objects) encountered during the shaft excavation operation, which prevents or hinders the advance of the shaft excavation. When efforts to advance past the obstruction to the design shaft tip elevation result in the rate of advance of the shaft drilling equipment being significantly reduced relative to the rate of advance for the rest of the shaft excavation, then the Contractor shall remove the obstruction under the provisions of Section 6-12.5. The method of removal of such obstructions, and the continuation of excavation shall be as proposed by the Contractor and accepted by the Engineer.

The Contractor shall use appropriate means to clean the bottom of the excavation of all shafts. No more than 2 inches of loose or disturbed material shall be present at the bottom of the shaft just prior to beginning concrete placement.

The Contractor shall not begin placing steel reinforcing bars and concrete in the shaft until receiving the Engineer's acceptance of the shaft excavation.

The steel reinforcing bar cage shall be rigidly braced to retain its configuration during handling and construction. The Contractor shall not place individual or loose bars. The Contractor shall install the steel reinforcing bar cage as specified in the shaft construction plan. The Contractor shall maintain the minimum concrete cover shown in the Plans.

If casings are used, the Contractor shall remove the casing during concrete placement. A minimum 5-feet head of concrete shall be maintained to balance soil and water pressure at the bottom of the casing. The casing shall be smooth. Where the top of the shaft is above the existing ground, the Contractor shall case the top of the hole prior to placing the concrete.

Concrete for shafts shall conform to Class 4000P. The Contractor shall place concrete in the shaft immediately after completing the shaft excavation and receiving the Engineer's acceptance of the excavation. The Contractor shall place the concrete in one continuous operation to the elevation shown in the Plans, using a method to prevent segregation of aggregates. The Contractor shall place the concrete as specified in the shaft construction plan. If water is present, concrete shall be placed in accordance with Section 6-02.3(6)B.

6-12.3(4) Trench, Grade Beam, or Spread Footing Construction

Where the noise barrier wall foundations exist below the existing ground line, excavation shall conform to Section 3-07.3(4), and to the limits and construction stages shown in the Plans. Foundation soils found to be unsuitable shall be removed and replaced in accordance with Section 3-07.3(1)C.

Where the noise barrier wall foundations exist above the existing ground line, the Contractor shall place and compact backfill material in accordance with Section 3-03.3(14)C.

Concrete for trench, grade beam, or spread footing foundations shall conform to Class 4000.

Cast-in-place concrete shall be formed, placed, and cured in accordance with Section 6-02, except that concrete for trench foundations shall be placed against undisturbed soil. Cast-in-place footings shall have a longitudinal slope no steeper than 1V: 6H, unless otherwise shown in the Plans or *Standard Plans*.

The excavation shall be backfilled in accordance with item 1 of the Compaction Subsection of Section 3-07.3(1)E.

The steel reinforcing bar cage and the noise barrier wall anchor bolts shall be installed and rigidly braced prior to grade beam and spread footing concrete placement to retain their configuration during concrete placement. The Contractor shall not place individual or loose steel reinforcing bars and anchor bolts, and shall not install anchor bolts during or after concrete placement.

6-12.3(5) Cast-In-Place Concrete Panel Construction

Construction of cast-in-place concrete panels for noise barrier walls shall conform to Section 6-11.3(4). For noise barrier walls with traffic barrier, the construction of the traffic barrier shall also conform to Section 6-10.3(2).

The top of the cast-in-place concrete panels shall conform to the top of wall profile shown in the Plans. Where a vertical step is constructed to provide elevation change between adjacent panels, the dimension of the step shall be 2 feet. Each horizontal run between steps shall be a minimum of 48 feet.

6-12.3(6) Precast Concrete Panel Fabrication and Erection

The Contractor shall construct the precast concrete panels in accordance with Section 6-02.3(9), and the following requirements:

1. Concrete shall conform to Class 4000.
2. Except as otherwise noted in the Plans and Special Provisions, all concrete surfaces shall receive a Class 2 finish in accordance with Section 6-02.3(14)B.
3. The Contractor shall fully support the precast concrete panel to avoid bowing and sagging surfaces.

   After receiving the Engineer's review of the shop drawings, the Contractor shall cast one precast concrete panel to be used as the sample panel. The Contractor shall construct the sample panel in accordance with the procedure and details specified in the shop drawings. The Contractor shall make the sample panel available to the Engineer for acceptance.

   Upon receiving the Engineer's acceptance of the sample panel, the Contractor shall continue production of precast concrete panels for the noise barrier wall. All precast concrete panels will be evaluated against the quality of the accepted sample panel. The sample panel shall be retained at the fabrication site until all precast concrete panels have been fabricated and accepted. After completing precast concrete panel fabrication, the Contractor may utilize the sample panel as a production noise barrier wall panel.
4. In addition to the fabrication tolerance requirements of Section 6-02.3(9), the precast concrete panels for noise barrier walls shall not exceed the following scalar tolerances:

   Length and Width: ± ⅛ inch per 5 feet, not to exceed ¼ inch total.
   Thickness: ± ¼ inch.

   The difference obtained by comparing the measurement of the diagonal of the face of the panels shall not be greater than ½ inch.

   Dimension tolerances for the traffic barrier portion of precast concrete panels formed with traffic barrier shapes shall conform to Section 6-10.3(2).
5. Precast concrete panels shall not be erected until the foundations for the panels have attained a minimum compressive strength of 3,400 psi.
6. The bolts connecting the precast concrete panels to their foundation shall be tightened to "snug tight" as defined in Section 6-03.3(32).
7. After erection, the precast concrete panels shall not exceed the joint space tolerances shown in the Plans. The panels shall not exceed ⅜ inch out of plumb in any direction.

   The Contractor shall seal the joints between precast concrete panels with a backer rod and sealant system as specified. The Contractor shall seal both sides of the joint full length.

The top of precast concrete panels shall conform to the top of wall profile shown in the Plans. Where a vertical step is constructed to provide elevation change between adjacent panels, the dimension of the step shall be 2 feet. Each horizontal run between steps shall be a minimum of 48 feet.

6-12.3(7) Masonry Wall Construction

Construction requirements for masonry noise barrier wall panels shall be as specified in the Special Provisions.

6-12.3(8) Fabricating and Erecting Timber Noise Barrier Wall Panels

Construction requirements for timber noise barrier wall panels shall be as specified in the Special Provisions.

6-12.3(9) Access Doors and Concrete Landing Pads

The Contractor shall install access doors and door frames as shown in the Plans and *Standard Plans*. The Contractor shall install the access doors to open toward the Roadway side. The door frames shall be set in place with grout conforming to either Section 9-20.3(2) or 9-20.3(4) and placed in accordance with Section 6-02.3(20), with the grout completely filling the void between the door frame and the noise barrier wall panel.

All door surfaces shall be painted in accordance with Section 6-07.3(9) except that primer shall be applied by the steel sheet supplier using methods and paint system as recommended by the manufacturer. Primer shall be compatible with the remaining coats as specified in Section 6-07.3(9). All primer coated exposed metal surfaces shall be field painted with the remaining Section 6-07.3(9)A paint system coats. The top coat, when dry, shall match the color specified in the Plans or Special Provisions.

The Contractor shall construct concrete landing pads for each access door location as shown in the Plans. The concrete shall conform to Section 6-02.3(2)B.

Access door deadbolt locks shall be capable of accepting a Best CX series core. The Contractor shall furnish and install a spring-loaded construction core lock with each lock. The Engineer will furnish the permanent Best CX series core for the Contractor to install at the conclusion of the project.

6-12.3(10) Finish Ground Line Dressing

The Contractor shall contour and dress the ground line on both sides of the noise barrier wall, providing the minimum cover over the foundation as shown in the Plans. The Contractor shall contour the ground adjacent to the barrier to ensure good drainage away from the barrier.

After the access roads are no longer needed for noise barrier wall construction activities, the Contractor shall restore the area to the original condition. The Contractor shall recontour the access roads to match into the surrounding ground and shall reseed all disturbed areas in accordance with the Section 8-01 and the Special Provisions, and the noise barrier wall access plan.

6-12.4 Measurement

Noise barrier wall will be measured by the square foot area of one face of the completed wall panel in place. Except as otherwise noted, the bottom limit for measurement will be the top of the trench footing, spread footing, or shaft cap. For Noise Barrier Type 5, the bottom measurement limit will be the optional construction joint at the base of the traffic barrier. For Noise Barrier Type 7, the bottom measurement limit will be base of the traffic barrier. For Noise Barrier Types 8, 11, 12, 14, 15, and 20, the bottom measurement limit will be the base of the wall panel.

Noise barrier wall access door will be measured once for each access door assembly with concrete landing pad furnished and installed.

6-12.5 Payment

Payment will be made for each of the following Bid items when they are included in the Proposal:

"Noise Barrier Wall Type __", per square foot.

The unit Contract price per square foot for "Noise Barrier Wall Type __" shall be full pay for constructing the noise barrier walls as specified, including constructing and removing access roads, excavating and constructing foundations and grade beams, constructing cast-in-place concrete, and masonry wall panels, fabricating and erecting precast concrete, and timber wall panels, applying sealer, and contouring the finish ground line adjacent to the noise barrier walls.

"Noise Barrier Wall Access Door", per each.

The unit Contract price per each for "Noise Barrier Wall Access Door" shall be full pay for furnishing and installing the access door assembly as specified, including painting the installed access door assembly and constructing the concrete landing pads.

"Removing Noise Barrier Wall Shaft Obstructions", estimated.

Payment for removing obstructions, as defined in Section 6-12.3(3), will be made for the changes in shaft construction methods necessary to remove the obstruction. The Contractor and the Engineer shall evaluate the effort made and reach agreement on the equipment and employees utilized, and the number of hours involved for each. Once these cost items and their duration have been agreed upon, the payment amount will be determined using the rate and markup methods specified in Section 1-09.6. For the purpose of providing a common proposal for all bidders, the Contracting Agency has entered an amount for the item "Removing Noise Barrier Wall Shaft Obstructions" in the bid proposal to become a part of the total bid by the Contractor.

If the shaft construction equipment is idled as a result of the obstruction removal work and cannot be reasonably reassigned within the project, then standby payment for the idled equipment will be added to the payment calculations. If labor is idled as a result of the obstruction removal work and cannot be reasonably reassigned within the project, then all labor costs resulting from Contractor labor agreements and established Contractor policies will be added to the payment calculations.

The Contractor shall perform the amount of obstruction work estimated by the Contracting Agency within the original time of the contract. The Engineer will consider a time adjustment and additional compensation for costs related to the extended duration of the shaft construction operations, provided:

1. The dollar amount estimated by the Contracting Agency has been exceeded, and;
2. The Contractor shows that the obstruction removal work represents a delay to the completion of the project based on the current progress schedule provided in accordance with Section 1-08.3.

6-13 Structural Earth Walls

6-13.1 Description

This Work consists of constructing structural earth walls (SEW).

6-13.2 Materials

Materials shall meet the requirements of the following sections:

Cement	9-01
Aggregates for Concrete	9-03.1
Gravel Borrow for Structural Earth Walls	9-03.14(4)
Premolded Joint Filler	9-04.1(2)
Steel Reinforcing Bar	9-07.2
Epoxy-Coated Steel Reinforcing Bar	9-07.3
Mortar	9-20.4
Concrete Curing Materials and Admixtures	9-23
Fly Ash	9-23.9
Water	9-25

Other materials required shall be as specified in the Special Provisions.

6-13.3 Construction Requirements

Proprietary structural earth wall systems shall be as specified in the Special Provisions.

6-13.3(1) Quality Assurance

The structural earth wall manufacturer shall provide a qualified and experienced representative to resolve wall construction problems. The structural earth wall manufacturer's representative shall be present at the beginning of wall construction activities, and at other times as needed throughout construction. Recommendations made by the structural earth wall manufacturer's representative shall be followed by the Contractor.

The completed wall shall meet the following tolerances:

1. Deviation from the design batter and horizontal alignment, when measured along a 10-foot straightedge, shall not exceed the following:
   a. Welded wire faced structural earth wall: 2 inches
   b. Precast concrete panel and concrete block faced structural earth wall: ¾ inch
2. Deviation from the overall design batter of the wall shall not exceed the following per 10 feet of wall height:
   a. Welded wire faced structural earth wall: 1½ inches
   b. Precast concrete panel and concrete block faced structural earth wall: ½ inch
3. The maximum outward bulge of the face between welded wire faced structural earth wall reinforcement layers shall not exceed 2 inches. The maximum allowable offset in any precast concrete facing panel joint shall be ¾ inch. The maximum allowable offset in any concrete block joint shall be ¾ inch.
4. The base of the structural earth wall excavation shall be within 3 inches of the staked elevations, unless otherwise accepted or specified by the Engineer.
5. The external structural earth wall dimensions shall be placed within 2 inches of that staked on the ground.
6. The backfill reinforcement layers shall be located horizontally and vertically within 1 inch of the locations shown in the structural earth wall Working Drawings.

At least 5 working days prior to the Contractor beginning any structural earth wall Work at the site, a structural earth wall preconstruction conference shall be held to discuss construction procedures, personnel, and equipment to be used, and other elements of structural earth wall construction. Those attending shall include:

1. (representing the Contractor) The superintendent, on site supervisors, and all foremen in charge of excavation, leveling pad placement, concrete block and soil reinforcement placement, and structural earth wall backfill placement and compaction.
2. (representing the Structural Earth Wall Manufacturer) The qualified and experienced representative of the structural earth wall manufacturer as specified at the beginning of this section.
3. (representing the Contracting Agency) The Engineer, key inspection personnel, and representatives from the WSDOT Construction Office and State Geotechnical Office.

6-13.3(2) Submittals

The Contractor, or the supplier as the Contractor's agent, shall furnish a Manufacturer's Certificate of Compliance certifying that the structural earth wall materials conform to the specified material requirements. This includes providing a Manufacturer's Certificate of Compliance for all concrete admixtures, cement, fly ash, steel reinforcing bars, reinforcing strips, reinforcing mesh, tie strips, fasteners, welded wire mats, backing mats, construction geotextile for wall facing, drainage geosynthetic fabric, block connectors, and joint materials. The Manufacturer's Certificate of Compliance for geogrid reinforcement shall include the information specified in Section 9-33.4(4) for each geogrid roll, and shall specify the geogrid polymer types for each geogrid roll.

A Type 1 Working Drawing of all test results performed by the Contractor or the Contractor's supplier, which are necessary to ensure compliance with the specifications, shall be submitted along with each Manufacturer's Certificate of Compliance.

Before fabrication, the Contractor shall submit a Type 1 Working Drawing consisting of the field construction manual for the structural earth walls, prepared by the wall manufacturer. This manual shall provide step-by-step directions for construction of the wall system.

The Contractor, through the license/patent holder for the structural earth wall system, shall submit Type 2E Working Drawings consisting of detailed design calculations and details. If not prepared by the license/patent holder for the structural earth system, the design calculation and working drawing submittal shall include documentation that the design calculation and working drawing submittal has been reviewed by, and received the concurrence of, the headquarters organization of the structural earth wall manufacturer as identified in the Special Provisions. Review and concurrence by a sales representative office is not acceptable.

6-13.3(2)A Design Calculation Content Requirements

The design calculation submittal shall include detailed design calculations based on the wall geometry and design parameters specified in the Plans and Special Provisions. The calculations shall include detailed explanations of any symbols, design input, materials property values, and computer programs used in the design of the walls. All computer output submitted shall be accompanied by supporting hand calculations detailing the calculation process. If MSEW 3.0, or a later version, is used for the wall design, hand calculations supporting MSEW are not required.

The design calculations shall be based on the current AASHTO *LRFD Bridge Design Specifications*, including current interims, the current WSDOT *Bridge Design Manual* LRFD (BDM), and the WSDOT *Geotechnical Design Manual* (GDM), and also based on the following:

1. The wall design calculations shall address all aspects of wall internal stability for the service, strength, and extreme event limit states.
2. The wall surcharge conditions (backfill slope) shown in the Plans.
3. If a highway is adjacent to and on top of the wall, a 2-foot surcharge shall be used in the design.
4. If the Plans detail SEW pedestrian barrier on top of the wall, the SEW pedestrian barrier shall be designed for a minimum TL-4 impact load, unless otherwise specified in the Plans or Special Provisions.
5. If the Plans detail a moment slab traffic barrier or SEW pedestrian barrier on top of the wall, the wall shall be designed for the impact load transferred from the barrier to the wall.
6. The geotechnical design parameters for the wall shall be as specified in the Special Provisions.
7. The soil reinforcement length shall be as shown in the Plans. If the Plans do not show a length, the length shall be either 6 feet or 0.7 times the wall design height H, whichever is greater.

If there are differences in design requirements between the AASHTO *LRFD Bridge Design Specifications* and the BDM or GDM, the BDM and GDM requirements shall govern.

6-13.3(2)B Working Drawing Content Requirements

All design details shown in the working drawings shall be selected from the design details and products specified for the specific structural earth wall manufacturer in the Preapproved Wall Appendix in the current WSDOT *Geotechnical Design Manual* (GDM). Geosynthetic reinforcement shown in the working drawings shall be selected from the products listed in the current WSDOT Qualified Products List (QPL). Substitution of design details and products not listed in the current WSDOT GDM or QPL will not be allowed.

The working drawing submittal shall include all details, dimensions, quantities, and cross sections necessary to construct the wall based on the wall geometry and design parameters specified in the Plans and Special Provisions, and shall include, but not be limited to, the following items:

1. A plan and elevation sheet or sheets for each wall, containing the following:
   a. An elevation view of the wall that includes the following:
      i. The elevation at the top of the wall, at all horizontal and vertical break points, and at least every 50 feet along the wall;
      ii. Elevations at the base of welded wire mats or the top of leveling pads and foundations, and the distance along the face of the wall to all steps in the welded wire mats, foundations, and leveling pads;
      iii. The designation as to the type of panel, block, or module;
      iv. The length, size, and number of geogrids or mesh or strips, and the distance along the face of the wall to where changes in length of the geogrids or mesh or strips occur; or
      v. The length, size, and wire sizes and spacing of the welded wire mats and backing mats, and the distance along the face of the wall to where changes in length, size, and wire sizes and spacing of the welded wire mats and backing mats occur; and
      vi. The location of the original and final ground line.

   b. A plan view of the wall that indicates the offset from the construction centerline to the face of the wall at all changes in horizontal alignment; the limit of the widest module, geogrid, mesh, strip, or welded wire mat, and the centerline of any drainage structure or drainage pipe that is behind or passes under or through the wall.
   c. General notes, if any, required for design and construction of the wall.
   d. All horizontal and vertical curve data affecting wall construction.
   e. A listing of the summary of quantities provided on the elevation sheet of each wall for all items, including incidental items.
   f. A cross section showing limits of construction. In fill sections, the cross section shall show the limits and extent of select granular backfill material placed above original ground.
   g. Limits and extent of reinforced soil volume.
2. All details, including steel reinforcing bar bending details. Bar bending details shall be in accordance with Section 9-07.1.
3. All details for foundations and leveling pads, including details for steps in the foundations or leveling pads.
4. All modules and facing elements shall be detailed. The details shall show all dimensions necessary to construct the element, all steel reinforcing bars in the element, and the location of reinforcement element attachment devices embedded in the precast concrete facing panel or concrete block.
5. All details for construction of the wall around drainage facilities, sign, signal, luminaire, and noise barrier wall foundations, and structural abutment and foundation elements and any other obstruction shall be clearly shown.
6. All details for connections to SEW pedestrian barriers, coping, parapets, noise barrier walls, and attached lighting shall be shown.
7. All details for the moment slab traffic barrier or SEW pedestrian barrier attached to the top of the wall (if shown in the Plans), including interaction with bridge approach slabs.

6-13.3(3) Excavation and Foundation Preparation

Excavation shall conform to Section 3-07.3(3). Foundation soils found to be unsuitable shall be removed and replaced in accordance with Section 3-07.3(1)C. The foundation for the Structure shall be graded level for a width equal to or exceeding the length of reinforcing as shown in the structural earth wall Working Drawings and, for walls with geogrid reinforcing, in accordance with Section 3-09.3. Prior to wall construction, the foundation, if not in rock, shall be compacted as accepted by the Engineer.

At the foundation level of the bottom course of precast concrete facing panels and concrete blocks, an unreinforced concrete leveling pad shall be provided as shown in the Plans. The leveling pad shall be cured a minimum of 12 hours and have a minimum compressive strength of 1,500 psi before placement of the precast concrete facing panels or concrete blocks.

6-13.3(4) Precast Concrete Facing Panel and Concrete Block Fabrication

Precast concrete facing panels shall conform to Section 6-02.3(9), except as modified in this section.

Concrete for precast concrete facing panels shall meet the following requirements:

1. Have a minimum 28-day compressive strength of 4,000 pounds per square inch, unless otherwise specified in the Special Provisions for specific proprietary wall systems.
2. Contain a water-reducing admixture meeting AASHTO M194 Type A, D, F, or G.
3. Be air-entrained, 6 percent ± 1½ percent.
4. Have a maximum slump of 4 inches, or 6 inches if a Type F or G water reducer is used.

Concrete for dry cast concrete blocks shall meet the following requirements:

1. Have a minimum 28-day compressive strength of 4,000 psi.
2. Conform to ASTM C1372, except as otherwise specified.
3. The lot of blocks produced for use in this project shall conform to the following freeze-thaw test requirements when tested in accordance with ASTM C1262. Minimum acceptable performance shall be defined as weight loss at the conclusion of 150 freeze-thaw cycles not exceeding 1 percent of the block's initial weight for a minimum of four of the five block specimens tested.
4. The concrete blocks shall have a maximum water absorption of 1 percent above the water absorption content of the lot of blocks produced and successfully tested for the freeze-thaw test specified in item 3 above.

The precast concrete facing panels and concrete blocks shall be considered acceptable regardless of curing age when compressive test results indicate that the compressive strength conforms to the 28-day requirements and when the visual inspection is satisfactorily completed. Testing of dry cast concrete blocks shall conform to ASTM C140.

All precast concrete facing panels shall be 5 feet square, except:

1. for partial panels at the top, bottom, and ends of the wall; and
2. as otherwise shown in the Plans.

All precast concrete facing panels shall be manufactured within the following tolerances:

1. All dimensions ± 3⁄16 inch.
2. Squareness, as determined by the difference between the two diagonals, shall not exceed ½ inch.
3. Surface defects on smooth formed surfaces measured on a length of 5 feet shall not exceed ⅛ inch. Surface defects on textured-finished surfaces measured on a length of 5 feet shall not exceed 5⁄16 inch.

All concrete blocks shall be manufactured within the following tolerances:

1. Vertical dimensions shall be ± 1⁄16 inch of the Plan dimension, and the rear height shall not exceed the front height.
2. The dimensions of the grooves in the top and bottom faces of the concrete blocks shall be formed within the tolerances specified by the proprietary wall manufacturer, for the fit required for the block connectors.
3. All other dimensions shall be ± ¼ inch of the Plan dimension.

Tie attachment devices, except for geosynthetic reinforcement, shall be set in place to the dimensions and tolerances shown in the Plans prior to casting.

The forms forming precast concrete facing panels, including the forms for loop pockets and access pockets, and the forms forming the concrete blocks, shall be removed in accordance with the recommendations of the wall manufacturer, without damaging the concrete.

The concrete surface for the precast concrete facing panel shall have the finish shown in the Plans for the front face and an unformed finish for the rear face. The rear face of the precast concrete facing panel shall be roughly screeded to eliminate open pockets of aggregate and surface distortions in excess of ¼ inch.

The concrete surface for the front face of the concrete block shall be flat, and shall be a conventional “split face” finish in accordance with the wall manufacturer’s specifications. The concrete surface of all other faces shall be Class 2 in accordance with Section 6-02.3(14)B. The finish and appearance of the concrete blocks shall also conform to ASTM C1372. The color of the concrete block shall be concrete gray, unless otherwise shown in the Plans.

The date of manufacture, production lot number, and the piece-mark, shall be clearly marked on the rear face of each precast concrete facing panel, and marked or tagged on each pallet of concrete blocks.

All precast concrete facing panels and concrete blocks shall be handled, stored, and shipped in accordance with Sections 6-02.3(9) to prevent chipping, cracks, fractures, and excessive bending stresses.

Precast concrete facing panels in storage shall be supported on firm blocking located immediately adjacent to tie strips to avoid bending the tie strips.

6-13.3(5) Precast Concrete Facing Panel and Concrete Block Erection

Precast concrete facing panels shall conform to Section 6-02.3(9), except as modified in this section.

The precast concrete facing panels shall be placed vertically. During erection, precast concrete facing panels shall be handled by means of a lifting device set into the upper edge of the panels.

Concrete blocks shall be erected in a running bond fashion in accordance with the wall manufacturer’s field construction manual, and may be placed by hand. The top surface of each course of concrete blocks, including all pockets and recesses, shall be cleaned of backfill and all extraneous materials prior to connecting the reinforcing strips or geosynthetic reinforcing, and placing the next course of concrete blocks. Concrete blocks receiving geosynthetic reinforcement shall be connected as specified in the Special Provisions. Cap block top courses shall be bonded to the lower course of concrete blocks as specified below. All other concrete blocks shall be connected with block connectors or pins placed into the connector slots.

Precast concrete facing panels and concrete blocks shall be placed in successive horizontal lifts as backfill placement proceeds in the sequence shown in the structural earth wall Working Drawings as approved by the Engineer.

External bracing is required for the initial lift for precast concrete facing panels.

As backfill material is placed behind the precast concrete facing panels, the panels shall be maintained in vertical position by means of temporary wooden wedges placed in the joint at the junction of the two adjacent panels on the external side of the wall.

Reinforcing shall be placed normal to the face of the wall, unless otherwise shown in the Plans or directed by the Engineer. Prior to placement of the reinforcing, backfill shall be compacted.

Geosynthetic reinforcing shall be placed in accordance with Section 3-09.3 and as follows:

1. The Contractor shall stretch out the geosynthetic in the direction perpendicular to the wall face to remove all slack and wrinkles, and shall hold the geosynthetic in place with soil piles or other methods as recommended by the geosynthetic manufacturer, before placing backfill material over the geosynthetic to the specified cover.

2. The geosynthetic reinforcement shall be continuous in the direction perpendicular to the wall face from the back face of the concrete panel to the end of the geosynthetic or to the last geogrid node at the end of the specified reinforcement length. Geosynthetic splices parallel to the wall face will not be allowed.

At the completion of each course of concrete blocks and prior to installing any block connectors or geosynthetic reinforcement at this level, the Contractor shall check the blocks for level placement in all directions, and shall adjust the blocks by grinding or rear face shimming, or other method as recommended by the structural earth wall manufacturer's representative and as approved by the Engineer, to bring the blocks into a level plane.

For concrete block wall systems receiving a cap block top course, the cap blocks shall be bonded to the lower course either with mortar conforming to Section 9-20.4(3), or with an adhesive capable of bonding the concrete block courses together.

6-13.3(6) Welded Wire Faced Structural Earth Wall Erection

The Contractor shall erect the welded wire wall reinforcement in accordance with the wall manufacturer's field construction manual. Construction geotextile for wall facing shall be placed between the backfill material within the reinforced zone and the coarse granular material immediately behind the welded wire wall facing, as shown in the Plans and the structural earth wall Working Drawings. Geosynthetic reinforcing, when used, shall be placed in accordance with Sections 3-09.3 and 6-13.3(5).

6-13.3(7) Backfill

Backfill placement shall closely follow erection of each course of welded wire mats and backing mats, precast concrete facing panels, or concrete blocks. Backfill shall be placed in such a manner as to avoid any damage or disturbance to the wall materials or misalignment of the welded wire mats and backing mats, precast concrete facing panels, or concrete blocks. Backfill shall be placed in a manner that segregation does not occur. Construction equipment shall not operate directly on the wall reinforcement. A minimum backfill thickness of 6 inches over the reinforcement shall be required prior to operation of vehicles or equipment.

The Contractor shall place wall backfill over geosynthetic reinforcement, or construction geotextile for wall facing, in accordance with Section 3-09.3.

Misalignment or distortion of the precast concrete facing panels or concrete blocks due to placement of backfill outside the limits of this Specification shall be corrected in a manner acceptable to the Engineer.

The moisture content of the backfill material prior to and during compaction shall be uniformly distributed throughout each layer of material. The moisture content of all backfill material shall conform to Sections 3-03.3(14)C and 3-03.3(14)D.

Backfill shall be compacted in accordance with Method C of Section 3-03.3(14)C, except as follows:

1. The maximum lift thickness after compaction shall not exceed 10 inches.
2. The Contractor shall decrease this lift thickness, if necessary, to obtain the specified density.
3. The Contractor shall not use sheepsfoot rollers or rollers with protrusions for compacting backfill reinforced with geosynthetic layers, or for compacting the first lift of backfill above the construction geosynthetic for wall facing for each layer of welded wire mats. Rollers shall have sufficient capacity to achieve compaction without causing distortion to the face of the wall in accordance with the tolerances specified in Section 6-13.3(1).

4. The Contractor shall compact the zone within 3 feet of the back of the wall facing panels without causing damage to or distortion of the wall facing elements (welded wire mats, backing mats, construction geotextile for wall facing, precast concrete facing panels, and concrete blocks) by using a plate compactor. No soil density tests will be taken within this area.
5. For wall systems with geosynthetic reinforcement, the minimum compacted backfill lift thickness of the first lift above each geosynthetic reinforcement layer shall be 6 inches.

At the end of each day's operation, the Contractor shall shape the last level of backfill to permit runoff of rainwater away from the wall face. In addition, the Contractor shall not allow surface runoff from adjacent areas to enter the wall construction site.

6-13.3(8) Guardrail Placement

Where guardrail posts are required, the Contractor shall not begin installing guardrail posts until completing the structural earth wall to the top of wall elevation shown in the Plans. The Contractor shall install the posts in a manner that prevents movement of the precast concrete facing panels or concrete blocks, and prevents ripping, tearing, or pulling of the wall reinforcement.

The Contractor may cut welded wire reinforcement of welded wire faced structural earth walls to facilitate placing the guardrail posts, but only in the top two welded wire reinforcement layers and only with the permission of the Engineer in a manner that prevents bulging of the wall face and prevents ripping or pulling of the welded wire reinforcement. Holes through the welded wire reinforcement shall be the minimum size necessary for the post. The Contractor shall demonstrate to the Engineer prior to beginning guardrail post installation that the installation method will not rip, tear, or pull the wall reinforcement.

The Contractor shall place guardrail posts between the reinforcing strips, reinforcing mesh, and tie strips of the non-geosynthetic reinforced precast concrete panel or concrete block faced structural earth walls. Holes through the reinforcement of geosynthetic reinforced walls, if necessary, shall be the minimum size necessary for the guardrail post.

6-13.3(9) Moment Slab Traffic Barrier and SEW Pedestrian Barrier

Moment slab traffic barrier and SEW pedestrian barrier shall be construction in accordance with Section 6-10.

6-13.4 Measurement

Structural earth wall will be measured by the square foot of completed wall in place. The bottom limits for vertical measurement will be the bottom of the bottom mat, for welded wire faced structural earth walls, or the top of the leveling pad (or bottom of wall if no leveling pad is present) for precast concrete panel or concrete block faced structural earth walls. The top limit for vertical measurement will be the top of wall as shown in the Plans. The horizontal limits for measurement are from the end of the wall to the end of the wall.

Gravel borrow for structural earth wall including haul will be measured by the cubic yard in place determined by the limits shown in the Plans.

6-13.5 Payment

Payment will be made for each of the following Bid items when they are included in the Proposal:

“Structural Earth Wall”, per square foot.

The unit Contract price per square foot for “Structural Earth Wall” shall be full payment for all costs to perform the Work in connection with constructing structural earth walls, including leveling pads and copings when specified.

“Gravel Borrow for Structural Earth Wall incl. Haul”, per cubic yard.

The unit Contract price per cubic yard for “Gravel Borrow for Structural Earth Wall incl. Haul” shall be full payment for all costs to perform the Work including, but not limited to, furnishing, hauling, placing and compacting the Gravel Borrow for Structural Earth Wall incl. Haul as measured. When required, all costs for Wall Facing Backfill shall be included in this unit contract item.

6-14 Geosynthetic Retaining Walls

6-14.1 Description

This Work consists of constructing geosynthetic retaining walls, as shown in the Plans.

6-14.2 Materials

Materials shall meet the requirements of the following sections:

Cement	9-01
Aggregates for Concrete	9-03.1
Sand	9-03.13(1)
Gravel Borrow for Structural Earth Wall	9-03.14(4)
Polyurethane Sealant	9-04.2(3)
Closed Cell Foam Backer Rod	9-04.2(3)A
Anchor Rods and Associated Nuts, Washers, and Couplers	9-06.5(4)
Reinforcing Steel	9-07
Welded Wire Reinforcement	9-07.7
Grout	9-20.3(4)
Construction Geosynthetic	9-33

Anchor plate shall conform to ASTM A36, ASTM A572 Grade 50, or ASTM A588.

The requirements specified in Section 3-09.2 for geotextile shall also apply to geosynthetic and geogrid materials used for permanent and temporary geosynthetic retaining walls.

Other materials required shall be as specified in the Special Provisions.

6-14.3 Construction Requirements

Temporary geosynthetic retaining walls are defined as those walls and wall components constructed and removed or abandoned before the Physical Completion Date of the project or as shown in the Plans. All other geosynthetic retaining walls shall be considered as permanent.

6-14.3(1) Tolerances

The completed wall(s) shall meet the following tolerances:

	Permanent Wall	Temporary Wall
Deviation from the design batter and horizontal alignment for the face when measured along a 10-foot straightedge at the midpoint of each wall layer shall not exceed:	±3 inches	±5 inches
Deviation from the overall design batter per 10 feet of wall height shall not exceed:	±2 inches	±3 inches
Maximum outward bulge of the face between backfill reinforcement layers shall not exceed:	±4 inches	±6 inches
Deviation from the staked elevation to the base of the retaining wall excavation:	±3 inches	±3 inches
Deviation from of staked ground to the external wall dimensions:	±2 inches	±2 inches
Deviation of the vertical reinforcement layers overlaps placement from that shown in the Plans:	±1 inches	±1 inches

6-14.3(2) Submittals

The Contractor shall submit Type 2E Working Drawings consisting of detailed plans for each wall. The Working Drawing submittal shall include all details, dimensions, and cross sections necessary to construct the wall based on the wall geometry and design parameters specified in the Plans, including at a minimum, the following:

1. Detailed wall plans, with plan and elevation views for each wall, showing the actual lengths proposed for the geosynthetic reinforcing layers and the locations of each geosynthetic product proposed for use in each of the geosynthetic reinforcing layers.
2. Detailed cross sections showing the geosynthetic reinforcing layers, fascia connection dowels or anchor rods, and fascia formwork connection or support details located within or adjacent to the wall backfill.
3. The Contractor's proposed wall construction method, including proposed forming systems, types of equipment to be used, proposed erection sequence and details of how the backfill will be retained during each stage of construction.
4. Manufacturer's Certificate of Compliance, samples of the retaining wall geosynthetic and sewn seams for the purpose of acceptance as specified.
5. Details of geosynthetic retaining wall corner construction, including details of the positive connection between the wall sections on both sides of the corner.
6. Details of terminating a top layer of retaining wall geosynthetic and backfill due to a changing retaining wall profile.

Acceptance of the Contractor's proposed wall construction details and methods shall not relieve the Contractor of their responsibility to construct the walls in accordance with the requirements of these Specifications.

6-14.3(3) Excavation and Foundation Preparation

Excavation shall conform to Section 3-07.3(3). Foundations soils found to be unsuitable shall be removed and replaced in accordance with Section 3-07.3(1)C.

The Contractor shall direct all surface runoff from adjacent areas away from the retaining wall construction site.

6-14.3(4) Erection and Backfill

The Contractor shall begin wall construction at the lowest portion of the excavation and shall place each layer horizontally as shown in the Plans. The Contractor shall complete each layer entirely before beginning the next layer.

Geotextile splices shall consist of a sewn seam or a minimum 1-foot overlap. Geogrid splices shall consist of adjacent geogrid strips butted together and fastened using hog rings, or other methods acceptable to the Engineer, in such a manner to prevent the splices from separating during geogrid installation and backfilling. Splices exposed at the wall face shall prevent loss of backfill material through the face. The splicing material exposed at the wall face shall be as durable and strong as the material to which the splices are tied. The Contractor shall offset geosynthetic splices in one layer from those in the other layers such that the splices shall not line up vertically. Splices parallel to the wall face will not be allowed, as shown in the Plans.

The Contractor shall stretch out the geosynthetic in the direction perpendicular to the wall face to ensure that no slack or wrinkles exist in the geosynthetic prior to backfilling.

For geogrids, the length of the reinforcement required as shown in the Plans shall be defined as the distance between the geosynthetic wrapped face and the last geogrid node at the end of the reinforcement in the wall backfill.

The Contractor shall place fill material on the geosynthetic in lifts such that 6 inches minimum of fill material is between the vehicle or equipment tires or tracks and the geosynthetic at all times. The Contractor shall remove all particles within the backfill material greater than 3 inches in size. Turning of vehicles on the first lift above the geosynthetic will not be permitted. The Contractor shall not end dump fill material directly on the geosynthetic without the prior permission of the Engineer.

The Contractor shall use a temporary form system to prevent sagging of the geosynthetic facing elements during construction. A typical example of a temporary form system and sequence of wall construction required when using this form are detailed in the Plans. Soil piles or the geosynthetic manufacturer's recommended method, in combination with the forming system shall be used to hold the geosynthetic in place until the specified cover material is placed.

The Contractor shall place and compact the wall backfill in accordance with the wall construction sequence detailed in the Plans and Method C of Section 3-03.3(14)C, except as follows:

1. The maximum lift thickness after compaction shall not exceed 10 inches
2. The Contractor shall decrease this lift thickness, if necessary, to obtain the specified density.
3. Rollers shall have sufficient capacity to achieve compaction without causing distortion to the face of the wall in accordance with Section 6-14.3(1).
4. The Contractor shall not use sheepsfoot rollers or rollers with protrusions.
5. The Contractor shall compact the zone within 3 feet of the back of the wall facing panels without causing damage to or distortion of the wall facing elements (welded wire mats, backing mats, construction geotextile for wall facing, precast concrete facing panels, and concrete blocks) by using a plate compactor. No soil density tests will be taken within this area.
6. For wall systems with geosynthetic reinforcement, the minimum compacted backfill lift thickness of the first lift above each geosynthetic reinforcement layer shall be 6 inches.

The Contractor shall construct wall corners at the locations shown in the Plans, and in accordance with the wall corner construction sequence and method in the Working Drawing submittal. Wall angle points with an interior angle of less than 150 degrees shall be considered to be a wall corner. The wall corner shall provide a positive connection between the sections of the wall on each side of the corner such that the wall backfill material cannot spill out through the corner at any time during the design life of the wall. The Contractor shall construct the wall corner such that the wall sections on both sides of the corner attain the full geosynthetic layer embedment lengths shown in the Plans.

Where required by retaining wall profile grade, the Contractor shall terminate top layers of retaining wall geosynthetic and backfill in accordance with the method in the Working Drawing submittal. The end of each layer at the top of the wall shall be constructed in a manner that prevents wall backfill material from spilling out the face of the wall throughout the life of the wall. If the profile of the top of the wall changes at a rate of 1:1 or steeper, this change in top of wall profile shall be considered to be a corner.

6-14.3(5) Guardrail Placement

The Contractor shall install guardrail posts as shown in the Plans after completing the wall, but before the permanent facing is installed. The Contractor shall install the posts in a manner that prevents bulging of the wall face and prevents ripping, tearing, or pulling of the geosynthetic reinforcement. Holes through the geosynthetic reinforcement shall be the minimum size necessary for the post. The Contractor shall demonstrate to the Engineer prior to beginning guardrail post installation that the installation method will not rip, tear, or pull the geosynthetic reinforcement.

6-14.3(6) Permanent Facing

The Contractor shall construct a permanent facing to the surface of all permanent geosynthetic retaining walls as shown in the Plans. Shotcrete facing, if shown in the Plans, shall conform to Section 6-18. Concrete fascia panel, if shown in the Plans, shall be constructed in accordance with Section 6-02. Cast-in-place concrete fascia panels shall be cured in accordance with the requirements in Section 6-02.3(11) for retaining walls. The Contractor shall apply the specified surface finish as noted, and to the limits shown, in the Plans to the exterior concrete surface. Precast concrete fascia panels shall conform to Section 6-02.3(9). When noted in the Plans, the Contractor shall apply pigmented sealer to the limits shown in the Plans.

Asphalt or cement concrete gutter shall be constructed as shown in the Plans and as specified in Section 8-04.

6-14.3(7) Moment Slab Traffic Barrier

Moment slab traffic barrier 42" single slope shall be constructed in accordance with Section 6-10.

6-14.4 Measurement

Permanent geosynthetic retaining wall and temporary geosynthetic retaining wall will be measured by the square foot of face of completed wall. Corner wrap area and extensions of the geosynthetic wall beyond the area of wall face shown in the Plans or staked by the Engineer are considered incidental to the wall construction and will not be included in the measurement of the square foot of face of completed geosynthetic retaining wall.

Gravel borrow for structural earth wall including haul will be measured by the cubic yard or ton in place determined by the limits shown in the Plans.

Shotcrete facing and concrete fascia panel will be measured by the square foot surface area of the completed facing or fascia panel, measured to the neat lines of the facing or panel as shown in the Plans. When a footing is required, the measurement of the fascia panel area will include the footing.

6-14.5 Payment

Payment will be made for each of the following Bid items when they are included in the Proposal:

> “Geosynthetic Retaining Wall”, per square foot.
>
> “Temporary Geosynthetic Retaining Wall”, per square foot.
>
> All costs in connection with constructing the temporary or permanent geosynthetic retaining wall as specified shall be included in the unit Contract price per square foot for “Geosynthetic Retaining Wall” and “Temporary Geosynthetic Retaining Wall”, including furnishing and installing the temporary forming system.

"Gravel Borrow for Structural Earth Wall Incl. Haul", per cubic yard.

The unit Contract price per cubic yard for "Gravel Borrow for Structural Earth Wall incl. Haul" shall be full payment for all costs to perform the Work including but not limited to furnishing, hauling, placing, and compacting the Gravel Borrow for Structural Earth Wall incl. Haul as measured.

"Concrete Fascia Panel For Geosynthetic Wall", per square foot.

All costs in connection with constructing the concrete fascia panels as specified shall be included in the unit Contract price per square foot for "Concrete Fascia Panel For Geosynthetic Wall", including all steel reinforcing bars, premolded joint filler, polyethylene bond breaker strip, joint sealant, PVC pipe for weep holes, exterior surface finish, and pigmented sealer (when specified), constructing and placing the concrete footing, edge beam, anchor beam, anchor rod assembly, and backfill.

Shotcrete facing will be paid for in accordance with Section 6-18.5.

6-15 Soil Nail Walls

6-15.1 Description

This Work consists of constructing soil nail walls.

6-15.2 Materials

Materials shall meet the requirements of the following sections:

Grout	9-20.3(4)
Prefabricated Drainage Mat	9-33.2(3)

Other materials required, including materials for soil nails, shall be as specified in the Special Provisions.

6-15.3 Construction Requirements

6-15.3(1) General Description

Soil nailing shall consist of excavating to the layer limits shown in the Plans, drilling holes at the specified angle into the native material, placing and grouting epoxy coated or encapsulated steel reinforcing bars (soil nails) in the drilled holes, placing prefabricated drainage material and steel reinforcement, and applying a shotcrete facing over the steel reinforcement. After completing the wall to full height, the Contractor shall construct the concrete fascia panels as shown in the Plans.

All proprietary items used in the soil nailed Structure shall be installed in accordance with the manufacturer’s recommendations. In the event of a conflict between the manufacturer’s recommendations and these Specifications, these Specifications shall prevail.

6-15.3(2) Contractor’s Experience Requirements

The Contractor or Subcontractor performing this Work shall have completed at least five projects, within the last 5 years, involving construction of retaining walls using soil nails or ground anchors or shall have completed the construction of two or more projects totaling at least 15,000 square feet of retaining wall with a minimum total of 500 soil nails or ground anchors.

The Contractor shall assign an engineer with at least 3 years of experience in the design and construction of permanently anchored or nailed Structures to supervise the Work. The Contractor shall not use consultants or manufacturer’s representatives in order to meet the requirements of this section. Drill operators and on-site supervisors shall have a minimum of 1 year experience installing permanent soil nails or ground anchors.

Contractors or Subcontractors that are specifically prequalified in Class 36 Work will be considered to have met the above experience requirements.

6-15.3(3) Submittals

The Contractor shall submit Type 2 Working Drawings of the following information.

1. A brief description of each project satisfying the Contractors Experience Requirements with the Owner’s name and current phone number (this item is not required if the Contractor or Subcontractor is prequalified in Class 36).
2. A list identifying the following personnel assigned to this project and their experience with permanently anchored or nailed Structures:
   a. Supervising Engineer.
   b. Drill Operators.
   c. On-site Supervisors who will be assigned to the project.

3. The proposed detailed construction procedure that includes:
    a. Proposed method(s) of excavation of the soil and/or rock.
    b. A plan for the removal and control of groundwater encountered during excavation, drilling, and other earth moving activities. Include a list of the equipment used to remove and control groundwater.
    c Proposed drilling methods and equipment.
    d. Proposed hole diameter(s).
    e. Proposed method of soil nail installation.
    f. Mix design and procedures for placing the grout.
    g. Shotcrete mix design with compressive strength test results.
    h. Procedures for placing the shotcrete (include placement in conditions when ground water is encountered).
    i. Encapsulation system for additional corrosion protection selected for the soil nails and anchorages requiring encapsulation.
4. Detailed Working Drawings of the method proposed for the soil nail testing that includes:
    a. All necessary drawings and details to clearly describe the proposed system of jacking support, framing, and bracing to be used during testing.
    b. Calibration data for each load cell, test jack, pressure gauge, stroke counter on the grout pump, and master gauge to be used. The calibration tests shall have been performed by an independent testing Laboratory, and tests shall have been performed within 60 calendar days of the date submitted. Testing or Work shall not commence until the Engineer has approved the load cell, jack, pressure gage, and master pressure gauge calibrations.
5. Certified mill test results and typical stress-strain curves along with samples from each heat, properly marked, for the soil nail steel. The typical stress-strain curve shall be obtained by approved standard practices. The guaranteed ultimate strength, yield strength, elongation, and composition shall be specified.

6-15.3(4) Preconstruction Conference

A soil nail preconstruction conference shall be held at least 5 working days prior to the Contractor beginning any permanent soil nail Work at the site to discuss construction procedures, personnel, materials and equipment to be used. Those attending shall include:

1. (representing the Contractor) The superintendent, on site supervisors, and all foremen in charge of excavating the soil face, drilling the soil nail hole, placing the soil nail and grout, placing the shotcrete facing, and tensioning and testing the soil nail.
2. (representing the Contracting Agency) The Engineer, key inspection personnel, and representatives from the WSDOT Construction Office and State Geotechnical Office.

If the Contractor's key personnel change, or if the Contractor proposes a significant revision of the approved permanent soil nail installation plan, an additional conference shall be held before any additional permanent soil nail operations are performed.

6-15.3(5) Earthwork

The ground contour above the wall shall be established to its final configuration and slope as shown in the Plans prior to beginning excavation of the soil for the first row of soil nails. All excavation shall conform to Section 3-03.

The excavation shall proceed from the top down in a horizontal lift sequence with the ground level excavated no more than 3 feet below the elevation of the row of nails to be installed in that lift. The excavated vertical wall face shall not be left unshored more than 24 hours for any reason. A lift shall not be excavated until the nail installation and reinforced shotcrete placement for the preceding lift has been completed and accepted. After a lift is excavated, the cut surface shall be cleaned of all loose materials, mud, rebound, and other foreign matter that could prevent or reduce shotcrete bond.

The accuracy of the ground cut shall be such that the required thickness of shotcrete can be placed within a tolerance of plus or minus 2 inches from the defined face of the wall, and over excavation does not damage overlying shotcrete sections by undermining or other causes.

The Contractor should review the geotechnical recommendations report prepared for this project for further information on the soil conditions at the location of each wall. Copies of the geotechnical recommendations report are available for review by prospective Bidders at the location identified in the Special Provisions.

6-15.3(6) Soil Nailing

The Contractor shall not handle and transport the encapsulated soil nails until the encapsulation grout has reached sufficient strength to resist damage during handling. The Contractor shall handle the encapsulated soil nails in such a manner to prevent large deflections or distortions during handling. When handling or transporting encapsulated soil nails, the Contractor shall provide slings or other equipment necessary to prevent damage to the soil nails and the corrosion protection. The Engineer may reject any encapsulated nails which are damaged during transportation or handling. Damaged or defective encapsulation shall be repaired in accordance with the manufacturer's recommendations.

Soil nails shall be handled and sorted in such a manner as to avoid damage or corrosion. Prior to inserting a soil nail in the drilled hole, the Contractor and the Engineer will examine the soil nail for damage. If, in the opinion of the Engineer, the epoxy coating or bar has been damaged, the nail shall be repaired. If, in the opinion of the Engineer, the damage is beyond repair, the soil nail shall be rejected.

If, in the opinion of the Engineer, the epoxy coating can be repaired, the Contractor shall patch the coating with an Engineer approved patching material.

Nail holes shall be drilled at the locations shown in the Plans or as staked by the Engineer. The nails shall be positioned plus or minus 6 inches from the theoretical location shown in the Plans. The Contractor shall select the drilling method and the grouting pressure used for the installation of the soil nail. The drill hole shall be located so that the longitudinal axis of the drill hole and the longitudinal axis of the nail are parallel. At the point of entry the soil nail shall be installed within plus or minus 3 degrees of the inclination from horizontal shown in the Plans, and the nail shall be within plus or minus 3 degrees of a line drawn perpendicular to the face of the wall unless otherwise shown in the Plans.

Water or other liquids shall not be used to flush cuttings during drilling, but air may be used. The nail shall be inserted into the drilled hole with centralizers to the desired depth in such a manner as to prevent damage to the drilled hole, sheathing or epoxy during installation. The centralizers shall provide a minimum of 0.5 inches of grout cover over the soil nail and shall be spaced no further than 8 feet apart. When the soil nail cannot be completely inserted into the drilled hole without difficulty, the Contractor shall remove the nail from the drilled hole and clean or redrill the hole to permit insertion. Partially inserted soil nails shall not be driven or forced into the hole. Subsidence, or any other detrimental impact from drilling shall be cause for immediate cessation of drilling and repair of all damages in a manner approved by the Engineer at no additional cost to the Contracting Agency.

If caving conditions are encountered, no further drilling will be allowed until the Contractor selects a method to prevent ground movement. The Contractor may use temporary casing. The Contractor's method to prevent ground movement shall be approved by the Engineer. The casings for the nail holes, if used, shall be removed as the grout is being placed.

Where necessary for stability of the excavation face, a sealing layer of shotcrete may be placed before drilling is started, or the Contractor shall have the option of drilling and grouting of nails through a stabilizing berm of native soil at the face of the excavation. The stabilizing berm shall extend horizontally from the soil face and from the face of the shotcrete a minimum distance of 1 foot, and shall be cut down from that point at a safe slope, no steeper than 1H:1V unless approved by the Engineer. The berm shall be excavated to final grade after installation and full length grouting of the nails. Nails damaged during berm excavation shall be repaired or replaced by the Contractor, to the satisfaction of the Engineer, at no added cost to the Contracting Agency.

If sections of the wall are constructed at different times than the adjacent soil nail sections, the Contractor shall use stabilizing berms, temporary slopes, or other measures acceptable to the Engineer, to prevent sloughing or failure of the adjacent soil nail sections.

If cobbles and boulders are encountered at the soil face during excavation, the Contractor shall remove all cobbles and boulders that protrude from the soil face into the design wall section and fill the void with shotcrete. All shotcrete used to fill voids created by removal of cobbles and boulders shall be incidental to shotcrete facing.

The grout equipment shall produce a grout free of lumps and undispersed cement. A positive displacement grout pump shall be used. The pump shall be equipped with a pressure gauge near the discharge end to monitor grout pressures. The pressure gauge shall be capable of measuring pressures of at least 150 psi or twice the actual grout pressures used by the Contractor, whichever is greater. The grouting equipment shall be sized to enable the grout to be pumped in one continuous operation. The mixer shall be capable of continuously agitating the grout.

The grout shall be injected from the lowest point of the drilled hole. The quantity of the grout and the grout pressures shall be recorded. The grout pressures and grout takes shall be controlled to prevent excessive ground heave.

The Contractor shall make and cure grout cubes once per day in accordance with WSDOT T 813. These samples shall be retained by the Contractor until all associated verification and proof testing of the soil nails has been successfully completed. If the Contractor elects to test the grout cubes for compressive strength, testing shall be conducted by an independent laboratory and shall be in accordance with the FOP for AASHTO T106.

6-15.3(7) Shotcrete Facing

Prior to placing shotcrete on an excavated layer, the Contractor shall vertically center prefabricated drainage mat between the columns of nails as shown in the Plans. The prefabricated drainage mat shall be installed in accordance with the manufacturer's recommendations. The permeable drain side shall be placed against the exposed soil face. The prefabricated drainage mat shall be installed after each excavation lift and shall be hydraulically connected with the prefabricated drainage mat previously placed, such that the vertical flow of water is not impeded. The Contractor shall tape all joints in the prefabricated drainage mat to prevent shotcrete intrusion during shotcrete application.

The Contractor shall place steel reinforcing bars and welded wire fabric and apply the shotcrete facing in accordance with Section 6-18 and the details shown in the Plans.

The shotcrete shall be constructed to the minimum thickness as shown in the Plans. Costs associated with additional thickness of shotcrete due to over excavation or irregularities in the cut face shall be borne by the Contractor.

Each soil nail shall be secured at the shotcrete facing with a steel plate as shown in the Plans. The plate shall be seated on a wet grout pad of a pasty consistency similar to that of mortar for bricklaying. The nut shall then be sufficiently tightened to achieve full bearing surface behind the plate. After the shotcrete and grout have had time to gain the specified strength, the nut shall be tightened with at least 100 foot-pounds of torque. After final tightening of the nut, the threads of the soil nail shall at a minimum be flush with the end of the nut.

6-15.3(8) Soil Nail Testing and Acceptance

Both verification and proof testing of the nails is required. The Contractor shall supply all materials, equipment, and labor to perform the tests. The Contractor shall submit Type 1 Working Drawings of all test data. Soil nails used for verification tests and proof tests shall not be production soil nails, but instead shall be separate sacrificial soil nails not otherwise incorporated into the Work.

The testing equipment shall include a dial gauge or vernier scale capable of measuring to 0.001 inch of the ground anchor movement. A hydraulic jack and pump shall be used to apply the test load. The movement-measuring device shall have a minimum travel equal to the theoretical elastic elongation of the total nail length plus 1 inch. The dial gauge or vernier scale shall be aligned so that its axis is within 5 degrees from the axis of the nail and shall be monitored with a reference system that is independent of the jacking system and excavation face.

The jack and pressure gauge shall be calibrated by an independent testing Laboratory as a unit. Each load cell, test jack and pressure gauge, grout pump stroke counter, and master gauge, shall be calibrated as specified in Section 6-15.3(3), item 4b. Additionally, the Contractor shall not use load cells, test jacks and pressure gauges, grout pump stroke counters, and master gauges, greater than 60 calendar days past their most recent calibration date, until such items are re-calibrated by an independent testing Laboratory.

The pressure gauge shall be graduated in increments of either 100 psi or 2 percent of the maximum test load, whichever is less. The pressure gauge shall be selected to place the maximum test load within the upper ⅓ of the range of the gauge. The ram travel of the jack shall not be less than the theoretical elastic elongation of the total length at the maximum test load plus 1 inch. The jack shall be independently supported and centered over the nail so that the nail does not carry the weight of the jack. The Contractor shall have a second calibrated jack pressure gauge at the site. Calibration data shall provide a specific reference to the jack and the pressure gauge.

The loads on the nails during the verification and proof tests shall be monitored to verify consistency of load – defined as maintaining the test load within 5 percent of the specified value. Verification and proof test loads less than 20,000 pounds or sustained for 5 minutes or less shall be monitored by the jack pressure gauge alone. Verification and proof test loads equal to or greater than 20,000 pounds and sustained for longer than 5 minutes shall be monitored with the assistance of an electric or hydraulic load cell. The Contractor shall provide the load cell, the readout device, and a calibration curve from the most recent calibration as specified in Section 6-15.3(3), item 4b. The load cell shall be selected to place the maximum test load within the middle ⅔ of the range of the load cell. The load cell shall be mounted between the jack and the anchor plate. The stressing equipment shall be placed over the nail in such a manner that the jack bearing plates, load cell and stressing anchorage are in alignment.

Nails to be tested shall be initially grouted no closer to the excavation face than the dimension shown in the Plans. After placing the grout, the nail shall remain undisturbed until the grout has reached strength sufficient to provide resistance during testing. Test nails shall be left in the ground after testing, with the exposed portion of the test nail cut and removed to 2 feet behind the excavated face or inside face of shotcrete. The drill holes for test nails shall be completely backfilled with grout or nonstructural filler after testing on those test nails has been completed.

Load testing shall be performed against a temporary reaction frame with bearing pads that bear directly against the existing soil or the shotcrete facing. Bearing pads shall be kept a minimum of 12 inches from the edges of the drilled hole and the load shall be distributed to prevent failure of the soil face or fracture of the shotcrete. The Contractor shall submit Type 2E Working Drawings of the reaction frame.

The soil nail load monitoring procedure for verification and proof test load greater than 20,000 pounds and sustained for longer than 5 minutes shall be as follows:

1. For each increment of load, attainment of the load shall be initially established and confirmed by the reading taken from the jack gauge.
2. Once the soil nail anchor load has been stabilized, based on the jack gauge reading, the load cell readout device shall immediately be read and recorded to establish the load cell reading to be used at this load. The load cell reading is intended only as a confirmation of a stable soil nail load and shall not be taken as the actual load on the soil nail.
3. During the time period that the load on the soil nail is held at this load increment, the Contractor shall monitor the load cell reading. The Contractor shall adjust the jack pressure as necessary to maintain the initial load cell reading. Jack pressure adjustment for any other reason will not be allowed.
4. Soil nail elongation measurements shall be taken at each load increment as specified in Sections 6-15.3(8)A and 6-15.3(8)B.
5. Steps 1 through 4 shall be repeated at each increment of load, in accordance with the load sequence specified in Sections 6-15.3(8)A and 6-15.3(8)B.

6-15.3(8)A Verification Testing

Verification testing shall be performed on nails installed within the pattern of production nails to verify the Contractor’s procedures, hole diameter, and design assumptions. No drilling or installation of production nails will be permitted in ground/rock unit unless successful verification testing of anchors in that unit has been completed and approved by the Engineer, using the same equipment, methods, nail inclination, nail length, and hole diameter as planned for the production nails. Changes in the drilling or installation method may require additional verification testing as determined by the Engineer and shall be done at no additional expense to the Contracting Agency. Verification tests may be performed prior to excavation for the soil nail wall.

Successful verification tests are required within the limits as specified in the Special Provisions. Test nail locations within these limits shall be at locations selected by the Engineer.

The Contractor shall submit Type 2E Working Drawings consisting of design details of the verification testing, including the system for distributing test load pressures to the excavation surface and appropriate nail bar size and reaction plate. The intent is to stress the bond between the grout and the surrounding soil/rock to at least twice the design load transfer. Prior to beginning verification testing, the Contractor shall measure and record the length of the nonbonded zone for each verification test soil nail.

The bar shall be proportioned such that the maximum stress at 200 percent of the test load does not exceed 80 percent of the yield strength of the steel. The jack shall be positioned at the beginning of the test such that unloading and repositioning of the jack during the test will not be required. The verification tests shall be made by incrementally loading the nails in accordance with the following schedule of hold time:

AL	1 minute
0.25TL	10 minutes
0.50TL	10 minutes
0.75TL	10 minutes
1.00TL	10 minutes
1.25TL	10 minutes
1.50TL	60 minutes
1.75TL	10 minutes
2.00TL	10 minutes

AL = Nail Alignment Load
TL = Nail Test Load

The test load shall be determined by the following equation = Test Load (TL) = Bond Length (BL) × Design Load Transfer (DLT).

The load shall be applied in increments of 25 percent of the test load. Each load increment shall be held for at least 10 minutes. Measurement of nail movement shall be obtained at each load increment. The load-hold period shall start as soon as the load is applied and the nail movement with respect to a fixed reference shall be measured and recorded at 1 minute, 2, 3, 4, 5, 6, 10, 20, 30, 40, 50, and 60 minutes.

The Engineer will evaluate the results of each verification test and make a determination of the suitability of the test and of the Contractor's proposed production nail design and installation system. Tests that fail to meet the design criteria will require additional verification testing or an approved revision to the Contractor's proposed production nail design and installation system. If a nail fails in creep, retesting will not be allowed.

A verification tested nail with a 60-minute load hold at 1.50TL is acceptable if:

1. The nail carries the test load with a creep rate that does not exceed 0.08 inch per log cycle of time and is at a linear or decreasing creep rate.
2. The total movement at the test load exceeds 80 percent of the theoretical elastic elongation of the non-bonded length.

Furthermore, a pullout failure shall not occur for the verification test anchor at the 2.0TL maximum load. Pullout failure load is defined as the load at which attempts to increase the test load result only in continued pullout movement of the test nail without a sustainable increase in the test load.

6-15.3(8)B Proof Testing

Proof tests shall be performed on proof test soil nails installed within the pattern of the production soil nails at the locations shown in the Plans. Proof test soil nails shall be installed using the same equipment, methods, nail inclination, nail length, and hole diameter as for adjacent production nails. The Contractor shall maintain the side-wall stability of the drill hole for the non-grouted portion during the test. The bond length shall be determined from the Nail Schedule and Test Nail Detail shown in the Plans. Prior to beginning proof testing, the Contractor shall measure and record the length of the nonbonded zone for each proof test soil nail.

Proof tests shall be performed by incrementally loading the nail in accordance with the schedule below. The anchor movement shall be measured and recorded to the nearest 0.001 inch with respect to an independent fixed reference point in the same manner as for the verification tests at the alignment load and at each increment of load. The load shall be monitored in accordance with Section 6-15.3(8). The scheduling of hold times shall be as follows:

AL	1 minute
0.25TL	5 minutes
0.50TL	5 minutes
0.75TL	5 minutes
1.00TL	5 minutes
1.25TL	5 minutes
1.50TL	10 minutes

AL = Nail Alignment Load
TL = Nail Test Load

The maximum load in a proof test shall be held for 10 minutes. The load hold period shall start as soon as the maximum load is applied and the nail movement with respect to an independent fixed reference shall be measured and recorded at 1, 2, 3, 4, 5, 6, and 10 minutes. The nail movement between 1 and 10 minutes shall not exceed 0.04 inches. If the nail movement between 1 and 10 minutes exceeds 0.04 inches, the maximum load shall be held an additional 50 minutes. If the load hold is extended, the nail movement shall be recorded at 20, 30, 40, 50, and 60 minutes. If a nail fails in creep, retesting will not be allowed.

A proof tested nail is acceptable if:

1. The nail carries the maximum load with less than 0.04 inches of movement between 1 and 10 minutes, unless the load hold extended to 60 minutes, in which case the nail would be acceptable if the creep rate does not exceed 0.08 inches per log cycle of time.
2. The total movement at the maximum load exceeded 80 percent of the theoretical elastic elongation of the non-bonded length.
3 The creep rate is not increasing with time during the load hold period.

If a proof test fails, the Engineer may direct the Contractor to replace some or all of the installed production nails between the failed test and an adjacent proof test nail that has met the test criteria. The Engineer may also require additional proof testing. All additional proof tests, and all installation of additional or modified nails, shall be performed at no additional expense to the Contracting Agency.

6-15.3(9) Concrete Fascia Panels

The Contractor shall construct the concrete fascia panels in accordance with Section 6-02 and the details in the Plans. The concrete fascia panels shall be cured in accordance with the Section 6-02.3(11) requirements specified for retaining walls. The Contractor shall provide the specified surface finish as noted, and to the limits shown, in the Plans to the exterior concrete surface. When noted in the Plans, the Contractor shall apply pigmented sealer to the limits shown in the Plans.

Asphalt or cement concrete gutter shall be constructed as shown in the Plans and as specified in Section 8-04.

6-15.4 Measurement

Prefabricated drainage mat will be measured by the square yard of material furnished and installed.

Soil nails will be measured per each for each soil nail installed and accepted.

Soil nail verification test and soil nail proof test will be measured per each for each successfully completed soil nail verification test and soil nail proof test at the locations specified in the Special Provisions and shown in the Plans.

Shotcrete facing and concrete fascia panel will be measured by the square foot surface area of the completed facing or fascia panel, measured to the neat lines of the facing or panel as shown in the Plans.

6-15.5 Payment

Payment will be made for each of the following Bid items when they are included in the Proposal:

“Soil Nail – Epoxy Coated”, per each.

“Soil Nail – Encapsulated”, per each.

All costs in connection with furnishing and installing the soil nails as specified shall be included in the unit Contract price per each for “Soil Nail - ___”, including all drilling, grouting, centralizers, bearing plates, welded shear connectors, nuts, and other Work required for installation of each soil nail.

“Prefabricated Drainage Mat”, per square yard.

“Soil Nail Verification Test and Soil Nail Proof Test”, per each.

All costs in connection with successfully completing soil nail verification tests and soil nail proof tests as specified shall be included in the unit contract price per each for “Soil Nail Verification Test and Soil Nail Proof Test”, including removal of the exposed portion of the test nail and backfilling the drilled hole with grout or nonstructural filler.

“Concrete Fascia Panel”, per square foot.

All costs in connection with constructing the concrete fascia panels as specified shall be included in the unit Contract price per square foot for “Concrete Fascia Panel”, including all steel reinforcing bars, premolded joint filler, polyethylene bond breaker strip, joint sealant, PVC pipe for weep holes, exterior surface finish, and pigmented sealer (when specified).

Shotcrete facing will be paid for in accordance with Section 6-18.5.

Unless otherwise specified, all costs in connection with excavation in front of the back face of the shotcrete facing shall be included in the unit Contract price per cubic yard for “Roadway Excavation” or “Roadway Excavation Incl. Haul” as specified in Section 3-03.5.

6-16 Soldier Pile and Soldier Pile Tieback Walls

6-16.1 Description

This Work consists of constructing soldier pile walls and soldier pile tieback walls.

6-16.2 Materials

Materials shall meet the requirements of the following sections:

Controlled Density Fill (CDF)	3-07.3(1)E
Lean Concrete	6-02.3(2)D
Cement	9-01
Aggregates for Concrete	9-03.1
Gravel Backfill	9-03.12
Premolded Joint Filler	9-04.1(2)
Welded Shear Studs	9-06.15
Steel Reinforcing Bar	9-07.2
Epoxy-Coated Steel Reinforcing Bar	9-07.3
Paints	9-08
Timber Lagging	9-09.2
Preservative Treatment for Timber Lagging	9-09.3(1)
Soldier Piles	9-10.5
Concrete Curing Materials and Admixtures	9-23
Fly Ash	9-23.9
Water	9-25
Prefabricated Drainage Mat	9-33.2(3)

Other materials required shall be as specified in the Special Provisions.

6-16.3 Construction Requirements

6-16.3(1) Quality Assurance

The steel soldier piles shall be placed so that the centerline of the pile at the top is within 1 inch of the Plan location. The steel soldier pile shall be plumb, to within 0.5 percent of the length based on the total length of the pile.

Welding, repair welding, and welding inspection shall conform to the Section 6-03.3(25) requirements for welding, repair welding, and welding inspection for all other steel fabrication.

6-16.3(2) Submittals

The Contractor shall submit Type 2 Working Drawings consisting of shop plans as specified in Section 6-03.3(7) for all structural steel, including the steel soldier piles, and shall submit Type 2 Working Drawings consisting of shop plans and other details as specified in Section 6-17.3(3) for permanent ground anchors.

The Contractor shall submit Type 1 Working Drawings consisting of the permanent ground anchor grout mix design and the procedures for placing the grout to the Engineer for approval.

The Contractor shall submit Type 2E Working Drawings consisting of forming plans for the concrete fascia panels, as specified in Sections 6-02.3(16) and 6-02.3(17).

1. Where the lateral pressure from concrete placement, as specified in Section 6-02.3(17)J, is less than or equal to the design earth pressure, the Contractor may tie forms directly to the soldier piles.

2. Where the lateral pressure from concrete placement, as specified in Section 6-02.3(17)J, is greater than the design earth pressure, the Contractor shall follow one of the following procedures:
    a. Tie the forms to strongbacks behind the lagging, or use some other system that confines the pressure from concrete placement between the lagging and the form panels, in addition to the ties to the soldier piles.
    b. Reduce the rate of placing concrete to reduce the pressure from concrete placement to less than or equal to the design earth pressure in addition to the ties to the soldier piles.
    c. Follow a procedure with a combination of a. and b.
3. The Contractor shall design the forms for an appropriate rate of placing concrete so that no cold joints occur, considering the wall thickness and height, and volume of concrete to be placed.

The Contractor shall submit Type 2 Working Drawings consisting of a shaft installation plan. In preparing the submittal, the Contractor shall reference the available subsurface data provided in the Contract test hole boring logs and the geotechnical report(s) prepared for this project. This plan shall provide at least the following information:

1. An overall construction operation sequence and the sequence of shaft construction.
2. List, description, and capacities of proposed equipment including but not limited to cranes, drills, augers, bailing buckets, final cleaning equipment, and drilling units. The narrative shall describe why the equipment was selected, and describe equipment suitability to the anticipated site and subsurface conditions. The narrative shall include a project history of the drilling equipment demonstrating the successful use of the equipment on shafts of equal or greater size in similar soil/rock conditions.
3. Details of shaft excavation methods including proposed drilling methods, methods for cleanout of the shafts, disposal plan for excavated material and drilling slurry (if applicable), and a review of method suitability to the anticipated site and subsurface conditions.
4. Details of the method(s) to be used to ensure shaft stability during excavation and concrete placement (e.g., using temporary casing, slurry, or other means to prevent caving or bottom heave). This shall include a review of method suitability to the anticipated site and subsurface conditions. If temporary casings are proposed, casing dimensions and detailed procedures for casing installation and removal shall be provided. If slurry is proposed, detailed procedures for mixing, using, maintaining, and disposing of the slurry shall be provided. A detailed mix design, and a discussion of its suitability to the anticipated subsurface conditions shall also be provided for the proposed slurry.
5. Details of soldier pile placement including internal support bracing and centralization methods.
6. Details of concrete placement including proposed operational procedures for pumping and/or tremie methods.
7. Details of the device used to prevent unauthorized entry into a shaft excavation.
8. The method to be used to form the horizontal construction joint at the top elevation specified for concrete Class 4000P in the shaft.

6-16.3(3) Shaft Excavation

Shafts shall be excavated to the required depth as shown in the Plans. The diameter of the shaft shall be as shown in the Plans. The excavation shall be completed in a continuous operation using equipment capable of excavating through the type of material expected to be encountered.

The Contractor may use temporary telescoping casing to construct the shafts.

If the shaft excavation is stopped the shaft shall be secured by installation of a safety cover. It shall be the Contractor's responsibility to ensure the safety of the shaft and surrounding soil and the stability of the sidewalls. A temporary casing, slurry, or other methods specified in the shaft installation plan shall be used if necessary to ensure such safety and stability.

Where caving in conditions are encountered, no further excavation will be allowed until the Contractor has implemented the method to prevent ground caving as submitted in accordance with item 4 of the Shaft Installation Plan.

No more than 2 inches of loose or disturbed material, for soldier piles with permanent ground anchors, nor more than 12 inches of loose or disturbed material, for soldier piles without permanent ground anchors, shall be present at the bottom of the shaft just prior to beginning concrete placement.

The excavated shaft shall be inspected and receive acceptance by the Engineer prior to proceeding with construction.

When obstructions are encountered, the Contractor shall notify the Engineer promptly. An obstruction is defined as a specific object (including, but not limited to, boulders, logs, and previously fabricated objects) encountered during the shaft excavation operation that prevents or hinders the advance of the shaft excavation. When efforts to advance past the obstruction to the design shaft tip elevation result in the rate of advance of the shaft drilling equipment being significantly reduced relative to the rate of advance for the rest of the shaft excavation, then the Contractor shall remove the obstruction under the provisions of Section 6-16.5. The method of removal of such obstructions, and the continuation of excavation shall be as proposed by the Contractor and approved by the Engineer.

Excavation of shafts shall not commence until a minimum of 12 hours after the shaft backfill for the adjacent shafts has been placed.

The temporary casings for the shafts shall be removed. A minimum 5-foot head of concrete shall be maintained to balance the soil and water pressure at the bottom of the casing. The casing shall be smooth.

6-16.3(4) Installing Soldier Piles

Soldier piles, if spliced, shall conform to all requirements of Section 6-05.3(6).

The prefabricated steel soldier piles shall be lowered into the drilled shafts and secured in position. Concrete cover over the soldier pile shall be 3 inches minimum, except that the cover over the soldier pile flange plate reinforcing at permanent ground anchor locations shall be 1½ inches minimum.

The steel soldier piles and attachments shall be shop painted after fabrication to the limits shown in the Plans with one coat of inorganic zinc primer. Application of the one coat of primer shall be in accordance with Section 6-07. The welded shear studs may be attached before or after painting. Paint damaged by welding shear studs in place does not require repair.

6-16.3(5) Backfilling Shaft

The excavated shaft shall be backfilled as shown in the Plans and subject to the following requirements:

1. Dry shaft excavations shall be backfilled with CDF or lean concrete.
2. Wet shaft excavations shall be backfilled with lean concrete, placed in accordance with the requirements for seal concrete in Section 6-02.3(6)B, except the minimum tube diameter shall be 4 inches.
3. Lean concrete shall meet the requirements of Section 6-02.3(2)D.
4. A wet shaft is defined as a shaft where water is entering the excavation and remains present to a depth of 6 inches or more.
5. When the Plans or test hole boring logs identify the presence of a water table at or above the elevation of the bottom of soldier pile shaft, the excavation shall be considered as wet, except as otherwise noted. Such a shaft may be considered a dry shaft provided the Contractor furnishes and installs casing that is sufficiently sealed into competent soils such that water cannot enter the excavation.

Placement of the shaft backfill shall commence immediately after completing the shaft excavation and receiving the Engineer's approval of the excavation. Vibration of shaft backfill is not required.

If water is not present, the shaft backfill shall be deposited by a method that prevents segregation of aggregates. The shaft backfill shall be placed such that the free-fall is vertical down the shaft without hitting the sides of the soldier pile or the excavated shaft. The Contractor's method for depositing the shaft backfill shall have approval of the Engineer prior to the placement of the shaft backfill.

6-16.3(6) Designing and Installing Lagging and Installing Permanent Ground Anchors

Lagging for soldier pile walls shall conform to one of the following two categories:

1. Temporary lagging is defined as lagging that is in service as a structural member for a maximum of 36 months before a permanent load-carrying fascia is in place, except for the following exception: Lagging for soldier pile walls in site soils conforming to an excluded soil type as defined under Section 6-16.3(6)A will be classified as permanent lagging conforming to Section 6-16.3(6)C, in which case this requirement will be specified in the Plans along with design details for such lagging.
2. Permanent lagging is defined as all lagging not conforming to the definition of temporary lagging as specified in category 1, above.

6-16.3(6)A Soil Classification

For the purposes of designing lagging for soldier pile walls, soils shall be categorized in the classifications defined below.

Soil Type 1

The following shall be considered Type 1 soils:

1. Cohesive fine-grained soils either CL or CH of medium consistency with $\gamma H/Su < 5$.
2. Cohesive fine-grained soils either CL or CH that are stiff to very stiff and nonfissured.
3. Fine-grained soils either ML or SM-ML that are above the water table.
4. Coarse-grained soils either GW, GP, GM, GC, SW, SP, or SM that are medium dense to dense.

Soil Type 2

The following shall be considered Type 2 soils:

1. Cohesive fine-grained soils either CL or CH that are heavily overconsolidated and fissured.
2. Fine-grained ML soils or coarse-grained SM-ML soils that are below the water table.
3. Coarse-grained SC soil that is medium dense to dense and is below the water table.
4. Coarse-grained soils either SW, SP, or SM that are loose.

Soil Type 3

The following shall be considered Type 3 soils:

1. Cohesive fine-grained soils CL and CH that are soft with $\gamma H/Su > 5$.
2. Fine-grained slightly plastic ML soil that is below the water table.
3. Coarse-grained SC soil that is loose and below the water table.

Exclusions

Regardless of whether site soils conform to one of the soil types defined above, site soils under the following conditions are excluded from the Type 1, Type 2, and Type 3 soil classifications:

1. Disturbed soils such as those in landslides or known unstable areas.
2. Layered soils dipping into the excavation steeper than 4H:1V.

Lagging for soldier pile walls located in site soils excluded from the Type 1, Type 2, and Type 3 soil classifications shall be designed in accordance with the latest AASHTO *LRFD Bridge Design Specifications* with current interim specifications. Use of the table in Section 6-16.3(6)B for timber lagging in these situations will not be allowed.

6-16.3(6)B Temporary Lagging

The Contractor shall design temporary lagging for all soldier pile walls. The temporary lagging design shall be based on the following:

1. The AASHTO *LRFD Bridge Design Specifications*, latest edition with current interim specifications, except that timber members used for temporary lagging may be selected based on the table below.
2. The soil type as specified in the Plans or as determined from the geotechnical report prepared for the project.
3. The soil pressure diagram, either as shown in the Plans or as included in the geotechnical report prepared for the project, including the surcharge for temporary construction load when shown in the Plans.

The Contractor shall submit Type 2E Working Drawings consisting of the soldier pile wall lagging design details and supporting design calculations. The submittal shall include, at a minimum, the following:

1. Description of the material used for the lagging, including identification of applicable material specifications.
2. Installation method and sequence.
3. If the lagging material is to be removed during or after installation of the permanent fascia, a description of how the lagging is removed without disturbing or damaging the fascia, soldier piles, and retained soil, and a description of how, and with what material, the void left by the removal of lagging is to be filled.

4. For all cases, except with timber for temporary lagging, a description with appropriate details of how subsurface drainage is to be accommodated, either in accordance with Section 6-16.3(7) for timber lagging, Section 6-15.3(7) for shotcrete facing, or other means appropriate for the geotechnical site conditions and acceptable to the Engineer for other lagging materials. Lagging materials and lagging installation methods that cause the buildup of, and prevent the relief of, pore water pressure will not be allowed. Free-draining materials are defined as those materials that exhibit a greater permeability than the material being retained.

Temporary lagging may be untreated timber conforming to the Section 9-09.2 requirements specified under Structures for timber lagging or another material selected by the Contractor.

Timber for temporary lagging shall conform to the minimum actual thickness specified in the table below for the soil type, exposed wall height, and lagging clear span as shown in the Plans.

Notwithstanding the requirements of Section 1-06.1, steel materials used by the Contractor as temporary lagging may be salvaged steel provided that the use of such salvaged steel materials shall be subject to visual inspection and acceptance by the Engineer. For salvaged steel materials where the grade of steel cannot be positively identified, the design stresses for the steel shall conform to the Section 6-02.3(17)B requirements for salvaged steel, regardless of whether rivets are present or not.

Minimum Actual Thickness of Timber Used as Temporary Lagging							
Soil Type[1]	**1**	**1**	**2**	**2**	**3**	**3**	**3**
Exposed Wall Height (feet)	**25 and under**	**Over 25 to 60**	**25 and under**	**Over 25 to 60**	**15 and under**	**Over 15 to 25**	**Over 25**
Clear Span of Lagging (feet)	**Minimum Actual Thickness of Rough Cut Timber Lagging (inches)[3]**						
5	2	3	3	3	3	3	4
6	3	3	3	3	3	4	5
7	3	3	3	4	4	5	6
8	3	4	4	4	5	6	See Note[2]
9	4	4	4	5	See Note[2]	See Note[2]	See Note[2]
10	4	5	5	5	See Note[2]	See Note[2]	See Note[2]

[1]Soil Type as defined in Section 6-16.3(6)A.
[2]For exposed wall heights exceeding the limits in the table above, or where minimum rough cut lagging thickness is not provided, the Contractor shall design the lagging in accordance with the latest AASHTO *LRFD Bridge Design Specifications* with current interim specifications.
[3]Table modified from FHWA document "Lateral Support Systems and Underpinning" (Report No. FHWA-RD-75-130).

6-16.3(6)C Permanent Lagging

Permanent lagging, including timber, shall be as shown in the Plans. The use of the table in Section 6-16.3(6)B for the design of timber lagging for permanent lagging will not be allowed.

6-16.3(6)D Installing Lagging and Permanent Ground Anchors

The excavation and removal of CDF and lean concrete from the shaft for the lagging installation shall proceed in advance of the lagging and shall not begin until the CDF or lean concrete in the shaft is of sufficient strength that the material remains in place during excavation and lagging installation. If the CDF or pumpable lean concrete separates from the soldier pile, or caves or spalls from around the soldier pile, the Contractor shall discontinue excavation and lagging installation operations until the CDF or lean concrete is completely set. The bottom of the excavation in front of the wall shall be level. Excavation shall conform to Section 3-03.

The lagging shall be installed from the top of the soldier pile proceeding downward. For walls without permanent ground anchors, the bottom of excavation at any one time shall not be more than 3 feet below the bottom level of the lagging installed up to that time, but in no case shall the depth of excavation beneath the bottom level of installed lagging be such to cause instability of the excavated face. For walls with permanent ground anchors, the bottom of excavation shall be not more than 3 feet below the permanent ground anchor level until all permanent ground anchors at that level are installed and stressed, but in no case shall the depth of excavation beneath the permanent ground anchor level be such to cause instability of the excavated face.

Installing, stressing, and testing the permanent ground anchors shall be in accordance with Section 6-17 and the construction sequence specified in the Plans.

6-16.3(6)E Backfill Behind Lagging

The lagging shall make direct contact with the soil. When and where lagging is not in full contact with the soil being retained, either the lagging shall be wedged back to create contact or the void shall be filled with a free-draining material. Caving that occurs during excavation shall be backfilled with free-draining material.

When utilizing lagging in fill situations, the backfill layers shall be placed in accordance with Section 3-03.3(14) except that all layers shall be compacted to 90 percent of maximum density.

6-16.3(7) Prefabricated Drainage Mat

For walls with concrete fascia panels, a 4-foot-wide strip of prefabricated drainage mat shall be installed full height of the concrete fascia panel, centered between soldier pile flanges, unless otherwise shown in the Plans.

The prefabricated drainage mat shall be attached to the lagging in accordance with the manufacturer's recommendations. The fabric side shall face the lagging. Splicing of the prefabricated drainage mat shall be in accordance with the manufacturer's recommendations.

The Contractor shall ensure the hydraulic connection of the prefabricated drainage mat to the previously installed material so that the vertical flow of water is not impeded.

The Contractor shall tape all joints in the prefabricated drainage mat to prevent concrete intrusion during concrete fascia panel construction.

6-16.3(8) Concrete Fascia Panel

The Contractor shall construct the concrete fascia panels as shown in the Plans, and in accordance with the forming plan. The concrete fascia panels shall be cured in accordance with the Section 6-02.3(11) requirements specified for retaining walls.

The Contractor shall provide the specified surface finish as noted, and to the limits shown, in the Plans to the exterior concrete surface. When noted in the Plans, the Contractor shall apply pigmented sealer to the limits shown in the Plans.

Asphalt or cement concrete gutter shall be constructed as shown in the Plans.

6-16.4 Measurement

Soldier pile shaft construction will be measured by the linear foot of shaft excavated below the top of ground line for the shaft, defined as the highest existing ground point within the shaft diameter.

Furnishing soldier pile will be measured by the linear foot of pile assembly specified in the Proposal, including adjustments to the Plan quantity made in accordance with Section 1-04.4.

Lagging will be measured by the square foot area of lagging installed. The quantity will be computed based on the vertical dimension from the highest lagging elevation to the lowest lagging elevation between each pair of adjacent soldier piles as the height dimension and the center-to-center spacing of the soldier piles as the length dimension.

Prefabricated drainage mat will be measured by the square yard of material furnished and installed.

Concrete fascia panel will be measured by the square foot surface area of the completed fascia panel, measured to the neat lines of the panel as shown in the Plans.

6-16.5 Payment

Payment will be made for each of the following Bid items when they are included in the Proposal:

“Shaft - ___ Diameter”, per linear foot.

All costs in connection with constructing soldier pile shafts shall be included in the unit Contract price per linear foot for “Shaft - ___ Diameter”, including shaft excavation, temporary casing if used, CDF, lean concrete, concrete Class 4000P, and installing the soldier pile assembly.

“Furnishing Soldier Pile - ___”, per linear foot.

All costs in connection with furnishing soldier pile assemblies shall be included in the unit Contract price per linear foot for “Furnishing Soldier Pile - ___”, including fabricating and painting the pile assemblies, and field splicing and field trimming the soldier piles. Payment will be made based on the quantity specified in the Proposal unless changes are made to this quantity in accordance with Section 1-04.4, in which case the quantity specified in the Proposal will be adjusted by the amount of the change and will be paid for in accordance with Section 1-04.4.

“Lagging”, per square foot.

All costs in connection with furnishing and installing lagging shall be included in the unit contract price per square foot for “Lagging”, including design of temporary lagging and filling voids behind the lagging with a free-draining material as approved by the Engineer.

“Prefabricated Drainage Mat”, per square yard.

“Concrete Fascia Panel”, per square foot.

All costs in connection with constructing the concrete fascia panels as specified shall be included in the unit Contract price per square foot for “Concrete Fascia Panel”, including all steel reinforcing bars, premolded joint filler, polyethylene bond breaker strip, joint sealant, PVC pipe for weep holes, exterior surface finish, and pigmented sealer (when specified).

Unless otherwise specified, all costs in connection with non-shaft excavation, including all excavation required for placement of timber lagging, shall be included in the unit Contract price per cubic yard for “Roadway Excavation” or “Roadway Excavation Incl. Haul” as specified in Section 3-03.5.

"Removing Soldier Pile Shaft Obstructions", estimated.

Payment for removing obstructions, as defined in Section 6-16.3(3), will be made for the changes in shaft construction methods necessary to remove the obstruction. The Contractor and the Engineer shall evaluate the effort made and reach agreement on the equipment and employees utilized, and the number of hours involved for each. Once these cost items and their duration have been agreed upon, the payment amount will be determined using the rate and markup methods specified in Section 1-09.6. For the purpose of providing a common proposal for all bidders, the Contracting Agency has entered an amount for the item "Removing Soldier Pile Shaft Obstructions" in the bid proposal to become a part of the total bid by the Contractor.

If the shaft construction equipment is idled as a result of the obstruction removal work and cannot be reasonably reassigned within the project, then standby payment for the idled equipment will be added to the payment calculations. If labor is idled as a result of the obstruction removal work and cannot be reasonably reassigned within the project, then all labor costs resulting from Contractor labor agreements and established Contractor policies will be added to the payment calculations.

The Contractor shall perform the amount of obstruction work estimated by the Contracting Agency within the original time of the contract. The Engineer will consider a time adjustment and additional compensation for costs related to the extended duration of the shaft construction operations, provided:

1. The dollar amount estimated by the Contracting Agency has been exceeded, and;
2. The Contractor shows that the obstruction removal work represents a delay to the completion of the project based on the current progress schedule provided in accordance with Section 1-08.3.

6-17 Permanent Ground Anchors

6-17.1 Description

This Work consists of constructing permanent ground anchors.

6-17.2 Materials

Materials required, including materials for permanent ground anchors, shall be as specified in the Special Provisions.

6-17.3 Construction Requirements

The Contractor shall select the ground anchor type and the installation method, and determine the bond length and anchor diameter. The Contractor shall install ground anchors that will develop the load indicated in the Plans and verified by tests specified in Sections 6-17.3(8)A, 6-17.3(8)B, and 6-17.3(8)C.

6-17.3(1) Definitions

Anchor Devices – The anchor head wedges or nuts that grip the prestressing steel.

Bearing Plate – The steel plate that evenly distributes the ground anchor force to the Structure.

Bond Length – The length of the ground anchor that is bonded to the ground and transmits the tensile force to the soil or rock.

Ground Anchor – A system, referred to as a tieback or as an anchor, used to transfer tensile loads to soil or rock. A ground anchor includes all prestressing steel, anchorage devices, grout, coatings, sheathings, and couplers if used.

Maintaining Consistency of Load – Maintaining the test load within 5 percent of the specified value.

Minimum Guaranteed Ultimate Tensile Strength (MUTS) – The minimum guaranteed breaking load of the prestressing steel as defined by the specified standard.

Tendon Bond Length – The length of the tendon that is bonded to the anchor grout.

Tendon Unbonded Length – The length of the tendon that is not bonded to the anchor grout.

Total Anchor Length – The unbonded length plus the tendon bond length.

6-17.3(2) Contractor Experience Requirements

The Contractor or Subcontractor performing this Work shall have installed permanent ground anchors for a minimum of 3 years. Prior to the beginning of construction, the Contractor shall submit a list containing at least five projects on which the Contractor has installed permanent ground anchors. A brief description of each project and a reference shall be included for each project listed. As a minimum, the reference shall include an individual’s name and current phone number.

The Contractor shall assign an engineer to supervise the Work with at least 3 years of experience in the design and construction of permanently anchored Structures. The Contractor shall not use consultants or manufacturer’s representatives in order to meet the requirements of this section. Drill operators and on-site supervisors shall have a minimum of 1 year experience installing permanent ground anchors.

Contractors or Subcontractors that are specifically prequalified in Class 36 Work will be considered to have met the above experience requirements.

The Contractor shall allow up to 15 calendar days for the Engineer’s review of the qualifications and staff as noted above. Work shall not be started on anchored wall systems nor materials ordered until approval of the Contractor’s qualifications are given.

6-17.3(3) Submittals

The Contractor shall submit Type 2E Working Drawings consisting of details and structural design calculations for the ground anchor system or systems intended for use.

The Contractor shall submit a Type 1 Working Drawing consisting of a detailed description of the construction procedure proposed for use.

The Contractor shall submit a Type 2 Working Drawing consisting of ground anchor schedule giving:

1. Ground anchor number
2. Ground anchor factored design load
3. Type and size of tendon
4. Minimum total bond length
5. Minimum anchor length
6. Minimum tendon bond length
7. Minimum unbonded length

The Contractor shall submit a Type 2 Working Drawing detailing the ground anchor tendon and the corrosion protection system. Include details of the following:

1. Spacers and their location
2. Centralizers and their location
3. Unbonded length corrosion protection system, including the permanent rubber seal between the trumpet and the tendon unbonded length corrosion protection and the transition between the tendon bond length and the unbonded tendon length corrosion protection.
4. Bond length corrosion protection system
5. Anchorage and trumpet
6. Anchorage corrosion protection system
7. Anchors using non-restressable anchorage devices

The Contractor shall submit Type 2 Working Drawings consisting of shop plans as specified in Section 6-03.3(7) for all structural steel, including the permanent ground anchors.

The Contractor shall submit Type 1 Working Drawings consisting of the mix design for the grout conforming to Section 9-20.3(4) and the procedures for placing the grout. The Contractor shall also submit the methods and materials used in filling the annulus over the unbonded length of the anchor.

The Contractor shall submit Type 2 Working Drawings consisting of the method proposed to be followed for the permanent ground anchor testing. This shall include all necessary drawings and details to clearly describe the method proposed.

The Contractor shall submit Type 2 Working Drawings consisting of calibration data for each load cell, test jack, pressure gauge and master pressure gauge to be used. The calibration tests shall have been performed by an independent testing Laboratory and tests shall have been performed within 60 calendar days of the date submitted.

6-17.3(4) Preconstruction Conference

A permanent ground anchor preconstruction conference shall be held at least 5 working days prior to the Contractor beginning permanent ground anchor Work at the site to discuss construction procedures, personnel, materials, and equipment to be used. Those attending shall include:

1. (representing the Contractor) The superintendent, on site supervisors, and all foremen in charge of drilling the ground anchor hole, placing the permanent ground anchor and grout, and tensioning and testing the permanent ground anchor.
2. (representing the Contracting Agency) The Engineer, key inspection personnel, and representatives from the WSDOT Construction Office and State Geotechnical Office.

If the Contractor's key personnel change, or if the Contractor proposes a significant revision of the approved permanent ground anchor installation plan, an additional conference shall be held before additional permanent ground anchor operations are performed.

6-17.3(5) Tendon Fabrication

The tendons can be either shop or field fabricated. The tendon shall be fabricated as shown in the shop plans.

The Contractor shall select the type of tendon to be used. The tendon shall be sized so the factored design load does not exceed 80 percent of the minimum guaranteed ultimate tensile strength of the tendon. In addition, the tendon shall be sized so the maximum test load does not exceed 80 percent of the minimum guaranteed ultimate tensile strength of the tendon.

The Contractor shall be responsible for determining the bond length and tendon bond length necessary to develop the factored design load indicated in the Plans in accordance with Sections 6-17.3(8)A, 6-17.3(8)B, and 6-17.3(8)C. The minimum bond length shall be 10 feet in rock and 15 feet in soil.

When the Plans require the tendon bond length to be encapsulated, the tendon bond length portion of the tendon shall be corrosion protected by encapsulating the tendon in a grout-filled PE or PVC tube as specified in Section 6-17.2 as supplemented in the Special Provisions. The tendons can be grouted inside the encapsulation prior to inserting the tendon in the drill hole or after the tendon has been placed in the drill hole. Expansive admixtures can be mixed with the encapsulation grout if the tendon is grouted inside the encapsulation while outside the drill hole. The tendon shall be centralized within the bond length encapsulation with a minimum of 0.20 inches of grout cover. Spacers shall be used along the tendon bond length of multi-element tendons to separate the elements of the tendon so the prestressing steel will bond to the encapsulation grout.

Centralizers shall be used to provide a minimum of 0.5 inches of grout cover over the tendon bond length encapsulation. Centralizers shall be securely attached to the encapsulation and the center-to-center spacing shall not exceed 10 feet. In addition, the upper centralizer shall be located a maximum of 5 feet from the top of the tendon bond length and the lower centralizer shall be located a maximum of 1 foot from the bottom of the tendon bond length.

The centralizer shall be able to support the tendon in the drill hole and position the tendon so a minimum of 0.5 inches of grout cover is provided and shall permit free flow of grout.

Centralizers are not required on encapsulated, pressure-injected ground anchor tendons if the ground anchor is installed in coarse grained soils (more than 50 percent of the soil larger than the number 200 sieve) using grouting pressures greater than 150 psi.

Centralizers are not required on encapsulated, hollow-stem-augered ground anchor tendons if the ground anchor is grouted through and the hole is maintained full of a stiff grout (8-inch slump or less) during extraction of the auger.

The minimum unbonded length of the tendon shall be the greater of 15 feet or that indicated in the Plans.

Corrosion protection of the unbonded length shall be provided by a sheath completely filled with corrosion inhibiting grease or grout. If grease is used under the sheath, provisions shall be made to prevent the grease from escaping at the ends of the sheath. The grease shall completely coat the tendon and fill the voids between the tendon and the sheath.

If the sheath is not fabricated from a smooth tube, a separate bond breaker shall be provided. The bond breaker shall prevent the tendon from bonding to the anchor grout surrounding the tendon unbonded length.

The total anchor length shall not be less than that indicated in the Plans or the approved Working Drawing submittal.

Anchorage devices shall be capable of developing 95 percent of the minimum guaranteed ultimate tensile strength of the prestressing steel tendon. The anchorage devices shall conform to the static strength requirements of Section 3.1 of the Post Tensioning Institute *Specification for Unbonded Single Strand Tendons, First Edition – 1993*.

Non-restressable anchorage devices may be used except where indicated in the Plans.

Restressable anchorages shall be provided on those ground anchors that require reloading. The post-tensioning supplier shall provide a restressable anchorage compatible with the post-tensioning system provided.

The bearing plates shall be sized so the bending stresses in the plate do not exceed the yield strength of the steel when a load equal to 95 percent of the minimum guaranteed ultimate tensile strength of the tendon is applied, and the average bearing stress on the concrete does not exceed that recommended in Section 3.1.3 of the Post Tensioning Institute *Specification for Unbonded Single Strand Tendons, First Edition – 1993*.

The trumpet shall have an inside diameter equal to or larger than the hole in the bearing plate. The trumpet shall be long enough to accommodate movements of the Structure during testing and stressing. For strand tendons with encapsulation over the unbonded length, the trumpet shall be long enough to enable the tendon to make a transition from the diameter or the tendon in the unbonded length to the diameter of the tendon at the anchor head without damaging the encapsulation. Trumpets filled with corrosion-inhibiting grease shall have a permanent rubber seal provided between the trumpet and the tendon unbonded length corrosion protection. Trumpets filled with grout shall have a temporary seal provided between the trumpet and the tendon unbonded length corrosion protection or the trumpet shall overlap the tendon unbonded length corrosion protection.

6-17.3(6) Tendon Storage and Handling

Tendons shall be handled and stored in such a manner as to avoid damage or corrosion. Damage to the prestressing steel as a result of abrasions, cut, nicks, welds and weld splatter will be cause for rejection by the Engineer. The prestressing steel shall be protected if welding is to be performed in the vicinity. Grounding of welding leads to the prestressing steel is forbidden. Prestressing steel shall be protected from dirt, rust, and deleterious substances. A light coating of rust on the steel is acceptable. If heavy corrosion or pitting is noted, the Engineer will reject the affected tendons.

The Contractor shall use care in handling and storing the tendons at the site. Prior to inserting a tendon in the drill hole, the Contractor and the Engineer will examine the tendon for damage to the encapsulation and the sheathing. If, in the opinion of the Engineer, the encapsulation is damaged, the Contractor shall repair the encapsulation in accordance with the tendon supplier's recommendations and as approved by the Engineer. If, in the opinion of the Engineer, the smooth sheathing has been damaged, the Contractor shall repair it with ultra high molecular weight polyethylene (PE) tape. The tape shall be spiral wound around the tendon so as to completely seal the damaged area. The pitch of the spiral shall ensure a double thickness at all points.

6-17.3(7) Installing Permanent Ground Anchors

The Contractor shall select the drilling method, the grouting procedure, and the grouting pressure used for the installation of the ground anchor.

When caving conditions are encountered, no further drilling will be allowed until the Contractor selects a method to prevent ground movement. The Contractor may use a temporary casing. The Contractor's method to prevent ground movement shall be submitted as a Type 2 Working Drawing. The casings for the anchor holes, if used, shall be removed. The drill hole shall be located so the longitudinal axis of the drill hole and the longitudinal axis of the tendon are parallel. The ground anchor shall not be drilled in a location that requires the tendon to be bent in order to enable the bearing plate to be connected to the supported Structure. At the point of entry the ground anchor shall be installed within plus or minus 3 degrees of the inclination from horizontal shown in the Plans or the Working Drawing submittal. The ground anchors shall not extend beyond the Right of Way limits.

The tendon shall be inserted into the drill hole to the desired depth. When the tendon cannot be completely inserted without difficulty, the Contractor shall remove the tendon from the drill hole and clean or redrill the hole to permit insertion. Partially inserted tendons shall not be driven or forced into the hole.

The Contractor shall use a grout conforming to Section 6-17.2 as supplemented in the Special Provisions.

The grout equipment shall produce a grout free of lumps and undispersed cement. A positive displacement grout pump shall be used. The pump shall be equipped with a pressure gauge near the discharge end to monitor grout pressures. The pressure gauge shall be capable of measuring pressures of at least 150 psi or twice the actual grout pressures used by the Contractor, whichever is greater. The grouting equipment shall be sized to enable the grout to be pumped in one continuous operation. The mixer shall be capable of continuously agitating the grout.

The grout shall be injected from the lowest point of the drill hole. The grout may be pumped through grout tubes, casing, or drill rods. The grout can be placed before or after insertion of the tendon. The quantity of the grout and the grout pressures shall be recorded. The grout pressures and grout takes shall be controlled to prevent excessive heave in soils or fracturing of rock formations.

The Contractor shall make and cure grout cubes once per day in accordance with WSDOT T 813. These samples shall be retained by the Contractor until all associated verification, performance and proof testing of the permanent ground anchors has been successfully completed. If the Contractor elects to test the grout cubes for compressive strength, testing shall be conducted by an independent laboratory and shall be in accordance with the FOP for AASHTO T 106.

After grouting, the tendon shall not be loaded for a minimum of 3 days.

No grout shall be placed above the top of the bond length during the time the bond length grout is placed. The grout at the top of the drill hole shall not contact the back of the Structure or the bottom of the trumpet. Except as otherwise noted, only nonstructural filler shall be placed above the bond length grout prior to testing and acceptance of the anchor. The Contractor may place structural grout above the bond length grout prior to testing and acceptance of the anchor subject to the following conditions:

1. The anchor unbonded length shall be increased by 8 feet minimum.
2. The grout in the unbonded zone shall not be placed by pressure grouting methods.

The corrosion protection surrounding the unbonded length of the tendon shall extend up beyond the bottom seal of the trumpet or 1 foot into the trumpet if no trumpet seal is provided. If the protection does not extend beyond the seal or sufficiently far enough into the trumpet, the Contractor shall extend the corrosion protection or lengthen the trumpet.

The corrosion protection surrounding the no load zone length of the tendon shown in the Plans shall not contact the bearing plate or the anchor head during testing and stressing. If the protection is too long, the Contractor shall trim the corrosion protection to prevent contact.

The bearing plate and anchor head shall be placed so the axis of the tendon and the drill hole are both perpendicular to the bearing plate within plus or minus 3 degrees and the axis of the tendon passes through the center of the bearing plate at the intersection of the trumpet and the bearing plate when fully seated with the alignment load.

The trumpet shall be completely filled with corrosion inhibiting grease or grout. Trumpet grease can be placed anytime during construction. Trumpet grout shall be placed after the ground anchor has been tested. The Contractor shall demonstrate to the Engineer that the procedure selected by the Contractor for placement of either grease or grout produces a completely filled trumpet.

All anchorages permanently exposed to the atmosphere shall be covered with a corrosion inhibiting grease-filled or grout-filled cover. The Contractor shall demonstrate to the Engineer that the procedures selected by the Contractor for placement of either grease or grout produces a completely filled cover. If the Plans require restressable anchorages, corrosion inhibiting grease shall be used to fill the anchorage cover and trumpet.

6-17.3(8) Testing and Stressing

Each ground anchor shall be tested. The test load shall be simultaneously applied to the entire tendon. Stressing of single elements of multi-element tendons will not be permitted. The Engineer will record test data.

The testing equipment shall consist of a dial gauge or vernier scale capable of measuring to 0.001 inch and shall be used to measure the ground anchor movement. The movement-measuring device shall have a minimum travel equal to the theoretical elastic elongation of the total anchor length plus 1 inch. The dial gauge or vernier scale shall be aligned so that its axis is within 5 degrees from the axis of the tieback. A hydraulic jack and pump shall be used to apply the test load. The jack and pressure gauge shall be calibrated by an independent testing Laboratory as a unit. Each load cell, test jack and pressure gauge, and master pressure gauge, shall be calibrated as specified in Section 6-17.3(3). Additionally, the Contractor shall not use load cells, test jacks and pressure gauges, and master pressure gauges, greater than 60 calendar days past their most recent calibration date, until such items are re-calibrated by an independent testing Laboratory.

The pressure gauge shall be graduated in increments of either 100 psi or 2 percent of the maximum test load, whichever is less. The pressure gauge will be used to measure the applied load. The pressure gauge shall be selected to place the maximum test load within the upper ⅓ of the range of the gauge. The ram travel of the jack shall not be less than the theoretical elastic elongation of the total anchor length at the maximum test load plus 1 inch. The jack shall be independently supported and centered over the tendon so that the tendon does not carry the weight of the jack. The Contractor shall have a second calibrated jack pressure gauge at the site. Calibration data shall provide a specific reference to the jack and the pressure gauge.

The loads on the tiebacks during the performance and verification tests shall be monitored to verify consistency of load as defined in Section 6-17.3(1). Performance test loads, and verification test loads when specified in the Special Provisions, sustained for 5 minutes or less, and all proof test leads, shall be monitored by the jack pressure gauge alone. Performance test loads, and verification test loads when specified in the Special Provisions, sustained for longer than 5 minutes shall be monitored with the assistance of an electric or hydraulic load cell. The Contractor shall provide the load cell and a readout device. The load cell shall be mounted between the jack and the anchor plate. The load cell shall be selected to place the maximum test load within the middle ⅔ of the range of the load cell. The stressing equipment shall be placed over the ground anchor tendon in such a manner that the jack, bearing plates, load cell and stressing anchorage are in alignment.

The permanent ground anchor load monitoring procedure for performance test loads, and verification test loads when specified in the Special Provisions, sustained for longer than 5 minutes shall be as follows:

1. For each increment of load, attainment of the load shall be initially established and confirmed by the reading taken from the jack gauge.
2. Once the permanent ground anchor load has been stabilized, based on the jack gauge reading, the load cell readout device shall immediately be read and recorded to establish the load cell reading to be used at this load. The load cell reading is intended only as a confirmation of a stable permanent ground anchor load, and shall not be taken as the actual load on the permanent ground anchor.
3. During the time period that the load on the permanent ground anchor is held at this load increment, the Contractor shall monitor the load cell reading. The Contractor shall adjust the jack pressure as necessary to maintain the initial load cell reading. Jack pressure adjustment for all other reasons will not be allowed.
4. Permanent ground anchor elongation measurements shall be taken at each load increment as specified in Sections 6-17.3(8)A and 6-17.3(8)B.
5. Steps 1 through 4 shall be repeated at each increment of load, in accordance with the load sequence specified in Sections 6-17.3(8)A and 6-17.3(8)B.

6-17.3(8)A Verification Testing

Verification tests will be required only when specified in the Special Provisions.

6-17.3(8)B Performance Testing

Performance tests shall be done in accordance with the following procedures.
Five percent of the ground anchors or a minimum of three ground anchors, whichever is greater, shall be performance tested. The Engineer shall select the ground anchors to be performance tested. The first production anchor shall be performance tested.

The performance test shall be made by incrementally loading and unloading the ground anchor in accordance with the following schedule, consistent with the Load Resistance Factor Design (LRFD) design method. The load shall be raised from one increment to another immediately after a deflection reading.

Performance Test Schedule
Load
AL
0.25FDL
AL
0.25FDL
0.50FDL
AL
0.25FDL
0.50FDL
0.75FDL
AL
0.25FDL
0.50FDL
0.75FDL
1.00FDL
AL
Jack to lock-off load

Where:

AL is the alignment load

FDL is the factored design load.

The maximum test load in a performance test shall be held for 10 minutes. The load-hold period shall start as soon as the maximum test load is applied and the anchor movement, with respect to a fixed reference, shall be measured and recorded at 1, 2, 3, 4, 5, 6, and 10 minutes. If the anchor movement between 1 and 10 minutes exceeds 0.04 inches, the maximum test load shall be held for an additional 50 minutes. If the load-hold is extended, the anchor movement shall be recorded at 20, 30, 40, 50, and 60 minutes. If an anchor fails in creep, retesting will not be allowed. All anchors not performance tested shall be proof tested.

6-17.3(8)C Proof Testing

Proof tests shall be performed by incrementally loading the ground anchor in accordance with the following schedule, consistent with the LRFD design method. The load shall be raised from one increment to another immediately after a deflection reading. The anchor movement shall be measured and recorded to the nearest 0.001 inches with respect to an independent fixed reference point at the alignment load and at each increment of load. The load shall be monitored with a pressure gauge. At load increments other than the maximum test load, the load shall be held just long enough to obtain the movement reading.

Proof Test Schedule
Load
AL
0.25FDL
0.50FDL
0.75FDL
1.00FDL
Jack to lock-off load

Where:

AL is the alignment load

FDL is the factored design load

The maximum test load in a proof test shall be held for 10 minutes. The load-hold period shall start as soon as the maximum test load is applied and the anchor movement with respect to a fixed reference shall be measured and recorded at 1, 2, 3, 4, 5, 6, and 10 minutes. If the anchor movement between 1 and 10 minutes exceeds 0.04 inches, the maximum test load shall be held of an additional 50 minutes. If the load-hold is extended, the anchor movements shall be recorded at 20, 30, 40, 50, and 60 minutes. If an anchor fails in creep, retesting will not be allowed.

6-17.3(9) Permanent Ground Anchor Acceptance Criteria

A performance or proof tested ground anchor with a 10 minute load hold is acceptable if the:

1. Ground anchor carries the maximum test load with less than 0.04 inches of movement between 1 and 10 minutes; and
2. Total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the tendon unbonded length.

A verification, performance or proof tested ground anchor with a 60-minute load hold is acceptable if the:

1. Ground anchor carries the maximum test load with a creep rate that does not exceed 0.08 inches/log cycle of time and is a linear or decreasing creep rate.
2. Total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the tendon unbonded length.

If the total movement of the ground anchors at the maximum test load does not exceed 80 percent of the theoretical elastic elongation of the tendon unbonded length, the Contractor shall replace the ground anchor at no additional cost to the Contracting Agency. Retesting of a ground anchor will not be allowed.

When a ground anchor fails, the Contractor shall modify the design, the construction procedures, or both. These modifications may include, but are not limited to, installing replacement ground anchors, modifying the installation methods, increasing the bond length or changing the ground anchor type. Modifications that requires changes to the Structure shall have prior approval of the Engineer. Modifications of design or construction procedures shall be at the Contractor's expense.

Upon completion of the test, the load shall be adjusted to the lock-off load indicated in the Plans and transferred to the anchorage device. The ground anchor may be completely unloaded prior to lock-off. After transferring the load and prior to removing the jack a lift-off reading shall be made. The lift-off reading shall be within 10 percent of the specified lock-off load.

If the load is not within 10 percent of the specified lock-off load, the anchorage shall be reset and another lift-off reading shall be made. This process shall be repeated until the desired lock-off load is obtained.

6-17.4 Measurement

Permanent ground anchors will be measured per each for each permanent ground anchor installed and accepted.

Permanent ground anchor performance tests will be measured per each for each anchor performance tested.

The permanent ground anchor verification testing program will not be measured but will be paid for on a lump sum basis.

6-17.5 Payment

Payment will be made for each of the following Bid items when they are included in the Proposal:

"Permanent Ground Anchor", per each.

All costs in connection with furnishing and installing permanent ground anchors shall be included in the unit Contract price per each for "Permanent Ground Anchor", including proof testing of the installed anchor as specified

"Permanent Ground Anchor Performance Test", per each.

"Permanent Ground Anchor Verification Test", lump sum.

6-18 Shotcrete

6-18.1 Description

This Work consists of constructing permanent shotcrete facing using the wet-mixing method as shown on the Plans. Shotcrete constructed as concrete slope protection shall be constructed in accordance with Section 8-16.

6-18.2 Materials

Materials shall meet the requirements of the following sections:

Cement	9-01.2(1)
Aggregates for Portland Cement Concrete	9-03.1
Premolded Joint Filler	9-04.1(2)
Steel Reinforcing Bar	9-07.2
Epoxy-Coated Steel Reinforcing Bar	9-07.3
Concrete Curing Materials and Admixtures	9-23
Fly Ash	9-23.9
Ground Granulated Blast Furnace Slag	9-23.10
Microsilica Fume	9-23.11
Water	9-25.1

Aggregate for shotcrete shall meet the following gradation requirements expressed as percentages by weight:

Sieve Size	Percent Passing
½ inch	100
⅜ inch	90 to 100
No. 4	70 to 85
No. 8	50 to 70
No. 16	35 to 55
No. 30	20 to 35
No. 50	8 to 20
No. 100	2 to 10
No. 200	0 to 2.5

6-18.3 Construction Requirements

6-18.3(1) Submittals

6-18.3(1)A Shotcrete Mix Design and Mix Design Test Panel Testing Results

The Contractor shall submit Type 2 Working Drawings prior to beginning the placement of production shotcrete. The submittal shall consist of the following:

1. Shotcrete mix design(s) in accordance with Section 6-02.3(2)A Contractor Mix Design except aggregate shall be as specified in Section 6-18.
2. All shotcrete core test results required for the Mix Design Test Panel(s).

6-18.3(1)B Preproduction Test Panel Testing Results

The Contractor shall submit Type 2 Working Drawings prior to beginning the placement of production shotcrete. The submittal shall consist of the following:

1. All shotcrete core test results required for the preproduction test panels verification of shotcrete properties.
2. All shotcrete core test results required for the preproduction test panels for verification of placement methods.

6-18.3(1)C Preproduction Test Panels for Verification of Surface Finish

The Contractor shall submit Type 2 Working Drawings prior to beginning construction of production shotcrete. The submittal shall consist of the following:

1. All test panel dimensions for verification of surface finish.
2. Planned method, equipment, means of access, joint formwork, and materials for placement, finishing and curing of each shotcrete facing specified.
3. A detailed construction sequence which includes order of operations and maximum timing between operations (including placing, flash coating, finishing, fogging, curing). The sequence will also include the anticipated crew size and production rate for the work.
4. Documentation of the certification of each nozzle operator placing permanent shotcrete facing. Nozzle operator shall be certified for the method and position required by the Plans.

6-18.3(2) Preconstruction Meeting

Prior to placing production shotcrete, the Contractor shall participate in a preconstruction meeting with the Engineer. At a minimum, attendance at this meeting shall include representatives from the Contractor, shotcrete subcontractor, and shotcrete supplier. Discussion will include shotcrete testing and acceptance, shotcrete production testing, placement and curing.

6-18.3(3) Shotcrete Testing

The Contractor shall be responsible for constructing test panels that are required for shotcrete sampling and testing. The Contractor shall retain a testing Laboratory to perform the tests required for Mix Design and Preproduction testing except surface finish. Testing Laboratories' equipment shall be calibrated within 1 year prior to testing and testers shall be either ACI certified or qualified in accordance with AASHTO R 18.

6-18.3(3)A Mix Design Test Panel

The Contractor shall construct mix design test panels for each mix design in accordance with ASTM C1140 and the following requirements:

1. The panels shall be of adequate size and thickness to complete all required testing.
2. The nozzle operators producing the panels do not need to be the same personnel who will be placing the permanent shotcrete facing.

Prior to shotcrete placement for the mix design test panels, the Contractor shall measure the air content of the freshly mixed shotcrete in accordance with WAQTC FOP for AASHTO T 152. The Contractor shall adjust the air content of the freshly mixed concrete in order to ensure that the hardened shotcrete meets spacing factor and minimum air content requirements or durability requirements.

The Contractor shall obtain cores from the mix design test panels in accordance with ASTM C1604.

The cores shall be tested as follows and shall meet the following criteria:

1. Shotcrete core diameters shall be at least 4 inches and have a L/D ratio of 2:1.
2. Determine shotcrete core density in accordance with AASHTO T 22.
3. Determine compressive strength in accordance with AASHTO T 22, except that the cores shall be cured in accordance with Standard Curing in a moist condition in accordance with AASHTO R 100. Minimum compressive strength shall be 4000 psi at 28 days.
4. Determine the chloride ion content in accordance with AASHTO T 260. Chloride ion content shall not exceed the limits of Section 6-02.3(2) for reinforced concrete.

5. Satisfy one of the following requirements:
   a. Determine the spacing factor and air content in accordance with ASTM C457. The maximum spacing factor shall be 0.010 inches and the minimum air content shall be 4%.
   b. Determine the durability factor using Method A after 300 cycles in accordance with AASHTO T161.The minimum durability factor shall be 90 percent. Test samples shall be obtained from shotcrete batches of a minimum of 3.0 cubic yards.

6-18.3(3)B Preproduction Testing

After meeting the mix design test panels performance requirements and prior to constructing the permanent shotcrete facing, the Contractor shall schedule and perform preproduction testing.

Preproduction test panels shall be prepared at the project site with the same method of shotcrete installation, finishing and curing to construct the permanent shotcrete facing. Prior to placement in the preproduction test panels, the shotcrete shall be tested for air content in accordance with WAQTC FOP for AASHTO T 152.

All nozzle operators constructing preproduction test panels shall have a current ACI shotcrete Nozzleman Certification. Each nozzle operator shall construct preproduction test panels for verification of shotcrete properties, for verification of placement methods and if specified in the Plans a test panel for surface finish. Only nozzle operators who have constructed acceptable preproduction test panels shall be allowed to place permanent shotcrete facing. When the preproduction test panels are rejected for strength, density, air entrainment or grade, a second panel may be prepared at the Contractor's option. When the second panel is rejected for strength, density, air entrainment or grade, the nozzle operator shall not be permitted to place permanent shotcrete facing.

6-18.3(3)B1 Preproduction Test Panels for Verification of Shotcrete Properties

One test panel shall be constructed for each mix design and each anticipated shooting orientation. Test panels shall be constructed in accordance with ASTM C1140. No reinforcing steel shall be included. The Contractor shall adjust the air content of the freshly mixed concrete in order to ensure that the hardened shotcrete meets spacing factor and minimum air content requirements.

At the completion of the curing period, the Contractor shall take at least six cores from each test panel in accordance with ASTM C1604.

The cores shall be tested as follows and meet the following criteria:

1. Shotcrete core diameters shall be at least 4 inches and have a L/D ratio of 2:1.
2. Three cores shall be measured for density in accordance with AASHTO T 22. Density of each core shall be a minimum of 95% of the average core density determined in the mix design test panel testing.
3. Three cores shall be measured for compressive strength in accordance with AASHTO T 22, except that the cores shall be cured in accordance with Standard Curing in a moist condition in accordance with AASHTO R 100. Minimum compressive strength at 28 days shall be 4000 psi.
4. The remaining three cores not measured for compressive strength shall have the air void system assessed in accordance with ASTM C457. Shotcrete shall have a maximum spacing factor of 0.010 inches and a minimum air content of 4%.

6-18.3(3)B2 Preproduction Test Panels for Verification of Placement Methods

One preproduction test panel shall be constructed for each combination of mix design, anticipated shooting orientation, and wall reinforcing layout. The test panels shall be constructed in accordance with ASTM C1140. The minimum test panel size shall be

48 inches by 48 inches. Test panels shall be constructed to the same thickness shown in the Plans and shall include the same reinforcing type, size and layout and shall have the same finish as specified for the permanent shotcrete facing.

At the completion of the curing period, the Contractor shall take three cores from each panel in accordance with ASTM C1604. Core diameters shall be at least 4 inches. Cores shall be taken at locations where reinforcing steel is present. These cores shall be visually graded as follows:

Grade 1 - Shotcrete specimens are solid; there are no laminations, sandy areas or voids. Small air voids with maximum diameter of ⅛ inch and maximum length of ¼ inch are normal and acceptable. Sand pockets or voids behind continuous reinforcing steel are unacceptable. The surface against the form or bond plane shall be sound, without sandy texture or voids.

Grade 2 - Shotcrete specimens shall have no more than two laminations or sandy areas with dimensions not to exceed ⅛ inch thick by 1 inch long. The height, width, and depth of voids shall not exceed ⅜ inch. Porous areas behind reinforcing steel shall not exceed ½ inch in any direction except along length of reinforcing steel. The surface against the form or bond plane shall be sound, without sandy texture or voids.

Grade 3 - Shotcrete specimens shall have no more than two laminations or sandy areas with dimensions exceeding 3⁄16 inch thick by 1-¼ inches long, or one major void, sand pocket, or lamination containing loosely bonded sand not to exceed ⅝ inch thick and 1-¼ inches in width. The surface against the form or bond plane may be sandy, with voids containing overspray to a depth of 1⁄16 inch.

Grade 4 - Core shall meet, in general, requirements of Grade 3 cores, but may have two major flaws such as described for Grade 3, or may have one flaw with maximum dimension of 1 inch perpendicular to the face of the core, with maximum width of 1-½ inches. The end of the core that was shot against the form may be sandy, with voids containing overspray to a depth of ⅛ inch.

Grade 5 - Core that does not meet criteria of core grades 1 through 4, by being of poorer quality, shall be classified as Grade 5.

For the purpose of qualifying the nozzle operator, the panel will be acceptable if all of the following are met:

1. The mean grade of the cores is 2.5 or less.
2. No core is graded at 4 or higher.

If the mean grade of the cores exceeds 2.5, the Contractor may take three additional cores and calculate a mean based on all six cores. If the mean grade of the six cores is 2.5 or less, the panel will be acceptable.

The measurements, scaled photographs of the cores and grading shall be submitted to the Engineer as a Type 2 Working Drawing. Cores shall be provided to the Engineer upon request.

6-18.3(3)B3 Preproduction Test Panels for Verification of Surface Finish

When specified in the Plans, the Contractor shall prepare a surface finish test panel to demonstrate the ability of each concrete finisher to achieve the specified surface finish. The Engineer will determine the acceptability of the panel surface finish by comparing it against the surface finish specified in the Contract.

Upon approval, the surface finish test panel will serve as a reference for qualifying additional concrete finishers and as a basis for accepting the surface finish of production shotcrete work.

6-18.3(3)C Production Testing

6-18.3(3)C1 Sampling and Testing Fresh Concrete

At the start of each day of production, the shotcrete will be tested in accordance with Section 6-02.3(5)G for temperature, consistency, and air content and will be sampled in accordance with Section 6-02.3(5)H. The Contractor shall provide curing boxes in accordance with Section 6-02.3(5)H.

The air content of the freshly mixed concrete shall be a minimum of 4%. The Contractor shall adjust the air content of the freshly mixed concrete in order to assure 4% minimum air content in the hardened shotcrete.

6-18.3(3)C2 Production Test Panels

The Contractor shall construct one unreinforced production test panel in accordance with ASTM C1140 for each day's production of shotcrete facing. The production test panel shall be constructed and cured on site using the same methods and curing that will be used to construct the permanent shotcrete facing. Following curing of the production test panel, three cores shall be taken by the Contractor in accordance with ASTM C1604. Core diameters shall be at least 4 inches. The Production cores shall be delivered to the Engineer for testing, and shall meet the following requirements:

1. The cores shall be measured for density in accordance with AASHTO T 22. Density of each core shall be a minimum of 95% of the average core density determined in the mix design test panel testing.
2. Cores shall be tested for compressive strength in accordance with AASHTO T 22, except the specimens shall be tested within 7 calendar days after coring. Minimum compressive strength at 28 days shall be 4000 psi. The Contractor shall coordinate with the Contracting Agency regarding transporting the cores to the laboratory for testing. The Contractor shall allow 4 calendar days for the Contracting Agency testing.

The remainder of the panels shall remain the property of the Contractor.

6-18.3(4) Vacant

6-18.3(5) Placing Wire Reinforcement

Reinforcement of the shotcrete shall be placed as shown in the Plans. The wire reinforcement shall be securely fastened to the steel reinforcing bars so that it will be 1 to 1.5 inches from the face of the shotcrete at all locations, unless otherwise shown in the Plans. Wire reinforcement shall be lapped 1.5 squares in all directions, unless otherwise shown in the Plans.

6-18.3(6) Alignment Control

The Contractor shall install non-corroding alignment wires and thickness control pins to establish thickness and plane surface. The Contractor shall install alignment wires at corners and offsets not established by formwork. The Contractor shall ensure that the alignment wires are tight, true to line, and placed to allow further tightening. The Contractor shall remove the alignment wires after facing construction is complete.

6-18.3(7) Shotcrete Application

The Contractor shall not place shotcrete that cannot be finished in the same shift.

The Contractor shall not apply shotcrete when the ambient air temperature rises above 86 degrees Fahrenheit. The Contractor may submit a request to apply shotcrete during hot weather (ambient temperatures above 86 degrees Fahrenheit), but shall submit hot-weather shotcrete procedures as a Type 3 Working Drawing to obtain the Engineer’s approval. The Working Drawing shall address any necessary means to control the temperature of the freshly placed concrete, prevent drying and shrinkage cracking, and ensure evaporative moisture loss is controlled.

Shotcrete shall not be placed on substrates below 41 degrees Fahrenheit.

Temperature and time for placement of shotcrete shall meet the requirement of Sections 6-02.3(4)D and 6-02.3(6)A.

A clean, dry supply of compressed air sufficient for maintaining adequate nozzle velocity for all parts for the Work and for simultaneous operation of a blow pipe for cleaning away rebound shall be always maintained. Thickness, method of support, air pressure, and rate of placement of shotcrete shall be controlled to prevent sagging or sloughing of freshly applied shotcrete.

The shotcrete shall be applied from the lower part of the area upwards. Surfaces to be shot shall be damp, but free of standing water.

The nozzles shall be held at an angle approximately perpendicular to the working face and at a distance that will keep rebound at a minimum and compaction will be maximized. Shotcrete shall emerge from the nozzle in a steady uninterrupted flow. If, for any reason, the flow becomes intermittent, the nozzle shall be diverted from the Work until a steady flow resumes.

Deficiencies observed during shotcrete application such as the following, shall constitute a cause for shotcrete rejection:

1. Failures to control and remove build-up of overspray and rebound;
2. Incomplete consolidation of shotcrete around reinforcing steel and embedments;
3. Incorporation of shadows, excessive voids, delaminations, sags or sloughing; and
4. Failures to apply shotcrete to the required line, grade and tolerance.

The Engineer will inspect the shotcrete for evidence of excessive plastic or drying shrinkage cracking, tears, sloughs or other deficiencies. Sounding or other nondestructive testing may be used to check for voids or delamination. The Engineer may also evaluate the in-place shotcrete as follows:

1. Extraction of cores from the in-place shotcrete at locations selected by the Engineer and evaluation of such cores for compliance with the specifications;
2. Sawcutting or coring to check the adequacy of encasement of reinforcing steel and embedments.

Surface defects shall be repaired as soon as possible after initial placement of the shotcrete. All shotcrete which lacks uniformity; which exhibits segregation, honeycombing, or lamination; or which contains any dry patches, slugs, voids, or sand pockets, shall be removed and replaced with fresh shotcrete by the Contractor, to the satisfaction of the Engineer at no cost to the Contracting Agency.

Construction joints in the shotcrete shall be uniformly tapered over a minimum distance of twice the thickness of the shotcrete layer. The surface of the joints shall be cleaned and thoroughly wetted before adjacent shotcrete is placed. Shotcrete shall be placed in a manner that provides a finish with uniform texture and color across the construction joint.

When the shotcrete is specified as the final fascia finish, the shotcrete shall be wet cured in accordance with Section 6-02.3(11). The Contractor shall keep the surface of the freshly placed shotcrete wet by fogging until the wet cure is applied. For temporary faces, the shotcrete shall be cured by applying a clear curing compound in accordance with Section 9-23.2. The curing compound shall be applied immediately after final gunning. Two coats of curing compound shall be applied to the shotcrete surface immediately after finishing.

If field inspection or testing indicates that any shotcrete produced fails to meet the requirements, the Contractor shall immediately modify procedures, equipment, or system, to produce specification material.

6-18.3(8) Shotcrete Finishing

When the shotcrete facing is an interim coating to be covered by a subsequent shotcrete coating or a cast-in-place concrete fascia, the Contractor shall strike off the surface of the shotcrete facing with a roughened surface as specified in Section 6-02.3(12). The grooves of the roughened surface shall be either vertical or horizontal.

The shotcrete face shall be finished using the alternative finish treatment shown in the Plans. The alternatives are as follows:

Alternative A – After the surface has taken its initial set (crumbling slightly when cut), the surface shall be broom finished to secure a uniform surface texture.

Alternative B – Shotcrete shall be applied in a thickness a fraction beyond the alignment wires and forms. The shotcrete shall stiffen to the point where the surface does not pull or crack when screeded with a rod or trowel. Excess material shall be trimmed, sliced, or scraped to true lines and grade. Alignment wires shall be removed and the surface shall receive a steel trowel finish, leaving a smooth uniform texture and color. Once the shotcrete has cured, pigmented sealer shall be applied to the shotcrete face. The shotcrete surface shall be completed to within a tolerance of ½ inch of true line and grade.

Alternative C – Shotcrete shall be hand-sculptured, colored, and textured to simulate the relief, jointing, and texture of the natural backdrop surrounding the facing. The ends and base of the facing shall transition in appearance as appropriate to more nearly match the color and texture of the adjoining Roadway fill slopes. This may be achieved by broadcasting fine and coarse aggregates, rocks, and other native materials into the final surface of the shotcrete while it is still wet, allowing sufficient embedment into the shotcrete to become a permanent part of the surface.

Alternative D (Heavy Nozzle Finish) – The heavy nozzle finish shall conform to Alternative B method except that after the alignment wires are removed, the surface shall be flashed and sealed to a heavy nozzle finish. The surface shall have an amplitude of $\frac{3}{16}$" and be uniform in texture and color.

6-18.4 Measurement

Shotcrete facing will be measured by the square foot surface area of the completed facing measured to the neat lines of the facing as shown in the Plans.

6-18.5 Payment

Payment will be made for each of the following Bid items when they are included in the Proposal:

"Shotcrete Facing", per square foot.

All costs in connection with shotcrete testing and constructing shotcrete facing as specified shall be included in the unit Contract price per square foot for "Shotcrete Facing".

6-19 Shafts

6-19.1 Description

This work consists of constructing the shafts, including concrete-filled steel tube (CFST) shafts, in accordance with the Plans, these Specifications, and as designated by the Engineer.

6-19.2 Materials

Materials shall meet the requirements of the following sections:

Cement	9-01
Aggregates for Concrete	9-03.1
Steel Reinforcing Bar	9-07.2
Epoxy-Coated Steel Reinforcing Bar	9-07.3
Curing Materials and Admixtures	9-23
Fly Ash	9-23.9
Ground Granulated Blast Furnace Slag	9-23.10
Microsilica Fume	9-23.11
Water for Concrete	9-25.1
Permanent Casing	9-36.1(1)
Temporary Casing	9-36.1(2)
Mineral Slurry	9-36.2(1)
Synthetic Slurry	9-36.2(2)
Water Slurry	9-36.2(3)
Steel Reinforcing Bar Centralizers	9-36.3
Access Tubes and Caps	9-36.4
Grout for Access Tubes	9-36.5

6-19.3 Construction Requirements

6-19.3(1) Quality Assurance

6-19.3(1)A Shaft Construction Tolerances

Shafts shall be constructed so that the center at the top of the shaft is within the following horizontal tolerances:

Shaft Diameter (feet)	Tolerance (inches)
Less than or equal to 2	3
Greater than 2 and less than 5	4
5 or larger	6

Shafts shall be within 1.5 percent of plumb. For rock excavation, allowable tolerance can be increased to 2 percent maximum.

During drilling or excavation of the shaft, the Contractor shall make frequent checks on the plumbness, alignment, and dimensions of the shaft. Deviations exceeding the allowable tolerances shall be corrected with a procedure approved by the Engineer.

Shaft steel reinforcing bar placement tolerances shall conform to Section 6-02.3(24)C. The elevation of the top of the reinforcing cage for drilled shafts shall be within +6 inches and -3 inches from the elevation shown in the Plans.

6-19.3(1)B Nondestructive Testing of Shafts

6-19.3(1)B1 Nondestructive Quality Assurance (QA) Testing of Shafts

Unless otherwise specified in the Special Provisions, the Contractor shall perform nondestructive QA testing of shafts, except for those constructed completely in the dry. Either crosshole sonic log (CSL) testing in accordance with ASTM D 6760 or thermal integrity profiling (TIP) testing in accordance with ASTM D 7949 shall be used.

6-19.3(1)B2 Nondestructive Quality Verification (QV) Testing of Shafts

The Contracting Agency may perform QV nondestructive testing of shafts that have been QA tested by the Contractor. The Contracting Agency may test up to ten percent of the shafts. The Engineer will identify the shafts selected for QV testing and the testing method the Contracting Agency will use.

The Contractor shall accommodate the Contracting Agency's nondestructive testing.

6-19.3(1)C Shaft Preconstruction Conference

A shaft preconstruction conference shall be held at least 5 working days prior to the Contractor beginning shaft construction work at the site to discuss construction procedures, personnel, and equipment to be used, and other elements of the approved shaft installation narrative as specified in Section 6-19.3(2)B. Those attending shall include:

1. (Representing the Contractor) – The superintendent, on site supervisors, and all foremen in charge of excavating the shaft, placing the casing and slurry as applicable, placing the steel reinforcing bars, and placing the concrete. If synthetic slurry is used to construct the shafts, the slurry manufacturer's representative or approved Contractor's employees trained in the use of the synthetic slurry shall also attend.
2. (Representing the Contracting Agency) – The Engineer, key inspection personnel, and representatives from the WSDOT Construction and Geotechnical Offices.

If the Contractor proposes a significant revision of the approved shaft installation narrative, as determined by the Engineer, an additional conference shall be held before additional shaft construction operations are performed.

6-19.3(2) Shaft Construction Submittal

The shaft construction submittal shall be comprised of the following four components: construction experience; shaft installation narrative; shaft slurry technical assistance; and nondestructive QA testing personnel. The submittals shall be Type 2 Working Drawings, except the shaft slurry technical assistance and nondestructive QA testing personnel submittals shall be Type 1.

6-19.3(2)A Construction Experience

The Contractor shall submit a project reference list to the Engineer for approval verifying the successful completion by the Contractor of at least three separate foundation projects with shafts of diameters and depths similar to or larger than those shown in the Plans, and ground conditions similar to those identified in the Contract. A brief description of each listed project shall be provided along with the name and current phone number of the project owner or the owner's Contractor.

The Contractor shall submit a list identifying the on-site supervisors and drill rig operators potentially assigned to the project to the Engineer. The list shall contain a brief description of each individual's experience in shaft excavation operations and placement of assembled steel reinforcing bar cages and concrete in shafts. The individual experience lists shall be limited to a single page for each supervisor or operator.

1. On-site supervisors shall have a minimum 2 years experience in supervising construction of shaft foundations of similar size (diameter and depth) and scope to those shown in the Plans, and similar geotechnical conditions to those described in the boring logs and summary of geotechnical conditions. Work experience shall be direct supervisory responsibility for the on-site shaft construction operations. Project management level positions indirectly supervising on-site shaft construction operations is not acceptable for this experience requirement.
2. Drill rig operators shall have a minimum of 1 year experience in construction of shaft foundations.

The Engineer may suspend the shaft construction if the Contractor substitutes unapproved personnel. The Contractor shall be fully liable for the additional costs resulting from the suspension of work, and no adjustments in contract time resulting from the suspension of work will be allowed.

6-19.3(2)B Shaft Installation Narrative

The Contractor shall submit a shaft installation narrative to the Engineer. In preparing the narrative, the Contractor shall reference the available subsurface data provided in the contract test hole boring logs, the Summary of Geotechnical Conditions provided in the Appendix to the Special Provisions, and the geotechnical report(s) prepared for this project. This narrative shall provide at least the following information:

1. Proposed overall construction operation sequence.
2. Description, size, and capacities of proposed equipment, including but not limited to, cranes, drills, auger, bailing buckets, final cleaning equipment, and drilling unit. The narrative shall describe why the equipment was selected, and describe equipment suitability to the anticipated site conditions and work methods. The narrative shall include a project history of the drilling equipment demonstrating the successful use of the equipment on shafts of equal or greater size in similar soil/rock conditions. The narrative shall also include details of shaft excavation and cleanout methods.
3. Details of the method(s) to be used to ensure shaft stability (i.e., prevention of caving, bottom heave, using temporary casing, slurry, or other means) during excavation (including pauses and stoppages during excavation) and concrete placement. If permanent casings are required, casing dimensions and detailed procedures for installation shall be provided.
4. A slurry mix design, including all additives and their specific purpose in the slurry mix, with a discussion of its suitability to the anticipated subsurface conditions, shall be submitted and include the procedures for mixing, using, and maintaining the slurry.

   A detailed plan for quality control of the selected slurry, including tests to be performed, test methods to be used, and minimum and/or maximum property requirements that must be met to ensure the slurry functions as intended, considering the anticipated subsurface conditions and shaft construction methods, in accordance with the slurry manufacturer’s recommendations and these Special Provisions shall be included. As a minimum, the slurry quality control plan shall include the following tests:

Property	Test Method
Density	Mud Weight (Density), API 13B-1, Section 1
Viscosity	Marsh Funnel and Cup, API 13B-1, Section 2.2
PH	Glass Electrode, pH Meter, or pH Paper
Sand Content	Sand, API 13B-1, Section 5

5. Description of the method used to fill or eliminate all voids below the top of shaft between the plan shaft diameter and excavated shaft diameter, when permanent casing is specified.
6. Details of concrete placement, including proposed operational procedures for pumping methods, and a sample uniform yield form to be used by the Contractor for plotting the approximate volume of concrete placed versus the depth of shaft for all shaft concrete placement (except concrete placement in the dry).
7. When shafts are constructed in water, the submittal shall include seal thickness calculations, seal placement procedure, and descriptions of provisions for casing shoring dewatering and flooding.
8. Description and details of the storage and disposal plan for excavated material and drilling slurry (if applicable).
9. Reinforcing steel shop drawings with details of reinforcement placement, including bracing, centering, and lifting methods, and the method to ensure the reinforcing cage position is maintained during construction, including use of bar boots and/or rebar cage base plates, and including placement of rock backfill below the bottom of shaft elevation, provided the conditions of Section 6-19.3(5)D are satisfied.

   The reinforcing steel shop drawings and shaft installation narrative shall include, at a minimum:

   a. Procedure and sequence of steel reinforcing bar cage assembly.
   b. The tie pattern, tie types, and tie wire gages for all ties on permanent reinforcing and temporary bracing.
   c. Number and location of primary handling steel reinforcing bars used during lifting operations.
   d. Type and location of all steel reinforcing bar splices.
   e. Details and orientation of all internal cross-bracing, including a description of connections to the steel reinforcing bar cage.
   f. Description of how temporary bracing is to be removed.
   g. Location of support points during transportation.
   h. Cage weight and location of the center of gravity.
   i. Number and location of pick points used for lifting for installation and for transport (if assembled off-site).
   j. Crane charts and a description and/or catalog cuts for all spreaders, blocks, sheaves, and chockers used to equalize or control lifting loads.
   k. The sequence and minimum inclination angle at which intermediate belly rigging lines (if used) are released.
   l. Pick point loads at 0, 45, 60, and 90 degrees and at all intermediate stages of inclination where rigging lines are engaged or slackened.
   m. Methods and temporary supports required for cage splicing.
   n. For picks involving multiple cranes, the relative locations of the boom tips at various stages of lifting, along with corresponding net horizontal forces imposed on each crane.
10. Methods and equipment used to clean the interior surfaces of the CFST permanent casing, including pressure flushing, brushing and scraping, prior to placing steel reinforcing bars and concrete.
11. Methods, equipment, weld details, and welding procedures for installing shear rings at the top of CFST permanent casing when shown in the Plans.

The Engineer will evaluate the shaft installation narrative for conformance with the Plans, Specifications, and Special Provisions, within the review time specified. If deemed necessary by the Engineer, a Shaft Installation Narrative Submittal Teleconference Meeting will be scheduled by the Contracting Agency following review of the Contractor's initial submittal of the narrative and prior to Contracting Agency's formal response to the initial submittal. Teleconference participants shall include the following:

1. (Representing the Contractor) – The superintendent, on-site supervisors, and other Contractor personnel involved in the preparation of the shaft installation narrative.
2. (Representing the Contracting Agency) – The Engineer, key inspection personnel, and representatives from the Materials Laboratory, State Geotechnical Office, and the WSDOT Construction Office.

6-19.3(2)C Shaft Slurry Technical Assistance

If slurry other than water slurry is used to construct the shafts, the Contractor shall provide or arrange for technical assistance in the use of the slurry as specified in Section 6-19.3(4)A. The Contractor shall submit the following to the Engineer:

1. The name and current phone number of the slurry manufacturer's technical representative assigned to the project, and the frequency of scheduled visits to the project site by the synthetic slurry manufacturer's representative.
2. The name(s) of the Contractor's personnel assigned to the project and trained by the slurry manufacturer in the proper use of the slurry. The submittal shall include a signed training certification letter from the slurry manufacturer for each trained Contractor's employee listed, including the date of the training.

6-19.3(2)D Nondestructive QA Testing Organization and Personnel

The Contractor shall submit the names of the testing organizations, and the names of the personnel who will conduct nondestructive QA testing of shafts. The submittal shall include documentation that the qualifications specified below are satisfied. For TIP testing, the testing organization is the group that performs the data analysis and produces the final report. The testing organizations and the testing personnel shall meet the following minimum qualifications:

1. The testing organization shall have performed nondestructive tests on a minimum of three deep foundation projects in the last two years.
2. Personnel conducting the tests for the testing organization shall have a minimum of one year experience in nondestructive testing and interpretation.
3. The experience requirements for the organization and personnel shall be consistent with the testing methods the Contractor has selected for nondestructive testing of shafts.
4. Personnel preparing test reports shall be a Professional Engineer, licensed under Title 18 RCW, State of Washington, and shall seal the report in accordance with WAC 196-23-020.

6-19.3(3) Shaft Excavation

Shafts shall be excavated to the required depth as shown in the Plans. Shaft excavation operations shall conform to this section and the shaft installation narrative.

Shaft excavation shall not be started until the Contractor has received the Engineer's acceptance for the reinforcing steel centralizers required when the casing is to be pulled during concrete placement.

Except as otherwise noted, the Contractor shall not commence subsequent shaft excavations until receiving the Engineer's acceptance of the first shaft, based on the results and analysis of the nondestructive testing for the first shaft. The Contractor may commence subsequent shaft excavations prior to receiving the Engineer's acceptance of the first shaft, provided the following condition is satisfied:

> The Engineer permits continuing with shaft construction based on the Engineer's observations of the construction of the first shaft, including, but not limited to, conformance to the shaft installation narrative in accordance with Section 6-19.3(2)B, and the Engineer's review of Contractor's daily reports and Inspector's daily logs concerning excavation, steel reinforcing bar placement, and concrete placement.

6-19.3(3)A Conduct of Shaft Excavation Operations

Once the excavation operation has been started, the excavation shall be conducted in a continuous operation until the excavation of the shaft is completed, except for pauses and stops as noted, using approved equipment capable of excavating through the type of material expected. Pauses during this excavation operation, except for casing splicing, tooling changes, slurry maintenance, and removal of obstructions, are not allowed.

Pauses, defined as momentary interruptions of the excavation operation, will be allowed only for casing splicing, tooling changes, slurry maintenance, and removal of obstructions. Shaft excavation operation interruptions not conforming to this definition shall be considered stops. Stops for uncased excavations (including partially cased excavations) shall not exceed 16 hours duration. Stops for fully cased excavations, excavations in rock, and excavations with casing seated into rock, shall not exceed 65 hours duration.

For stops exceeding the time durations specified above, the Contractor shall stabilize the excavation using one or both of the following methods:

1. For an uncased excavation, before the end of the work day, install casing in the hole to the depth of the excavation. The outside diameter of the casing shall not be smaller than 6 inches less than either the plan diameter of the shaft or the actual excavated diameter of the hole, whichever is greater. Prior to removing the casing and resumption of shaft excavation, the annular space between the casing and the excavation shall be sounded. If the sounding operation indicates that caving has occurred, the casing shall not be removed and shaft excavation shall not resume until the Contractor has stabilized the excavation in accordance with the shaft installation narrative conforming to Section 6-19.3(2)B, item 3.
2. For both a cased and uncased excavation, backfill the hole with either CDF or granular material. The Contractor shall backfill the hole to the ground surface, if the excavation is not cased, or to a minimum of 5 feet above the bottom of casing (temporary or permanent), if the excavation is cased. Backfilling of shafts with casing fully seated into rock, as determined by the Engineer, will not be required.

During stops, the Contractor shall stabilize the shaft excavation to prevent bottom heave, caving, head loss, and loss of ground. The Contractor bears full responsibility for selection and execution of the method(s) of stabilizing and maintaining the shaft excavation, in accordance with Section 1-07.13. Shaft stabilization shall conform to the shaft installation narrative in accordance with Section 6-19.3(2)B, item 3.

If slurry is present in the shaft excavation, the Contractor shall conform to the requirements of Section 6-19.3(4)B of this Special Provision regarding the maintenance of the slurry and the minimum level of drilling slurry throughout the stoppage of the shaft excavation operation, and shall recondition the slurry to the required slurry properties in accordance with Section 9-36.2 prior to recommencing shaft excavation operations.

6-19.3(3)B Temporary and Permanent Shaft Casing

The Contractor shall furnish and install required temporary and permanent shaft casings as shown in the Plans and as specified in the Special Provisions.

6-19.3(3)B1 General Shaft Casing Requirements

Shaft casing shall be watertight and clean prior to placement in the excavation.

The outside diameter of the casing shall not be less than the specified diameter of the shaft, except when metric casing is specified for 4, 5, and 10 foot nominal shaft diameters, the outside diameter of the casing shall not be less than the specified diameter of the shaft minus 2 inches. The inside diameter of the casing shall not be greater than the specified diameter of the shaft plus 6 inches, except as otherwise noted for shafts 5 feet or less in diameter, and as otherwise noted in Section 6-19.3(3)B4 for temporary telescoping casing. The inside diameter of casings for shafts 5 feet or less in diameter shall not be greater than the specified diameter of the shaft plus 1 foot.

6-19.3(3)B2 Permanent Shaft Casing

Permanent casing is defined as casing designed as part of the shaft structure and installed to remain in place after construction is complete. All permanent casing shall be of ample strength to resist damage and deformation from transportation and handling, installation stresses, and all pressures and forces acting on the casing. Where the minimum thickness of permanent casing is specified in the Plans, it is specified to satisfy structural design requirements only. The Contractor shall increase the casing thickness as necessary to satisfy the requirements of this section.

For permanent casing for CFST shafts, see Section 6-19.3(3)J.

6-19.3(3)B3 Temporary Shaft Casing

Temporary casing is defined as casing installed to facilitate shaft construction only. Temporary casing is not designed as part of the shaft structure, and shall be completely removed after shaft construction is complete unless otherwise shown in the Plans. All temporary casing shall be of ample strength to resist damage and deformation from transportation and handling, installation and extraction stresses, and all pressures and forces acting on the casing. The casing shall be capable of being removed without deforming and causing damage to the completed shaft and without disturbing the surrounding soil.

To maintain stable excavations and to facilitate construction, the Contractor may furnish and install temporary casing in addition to the required casing specified in the Special Provisions. The Contractor shall provide temporary casing at the site in sufficient quantities to meet the needs of the anticipated construction method.

6-19.3(3)B4 Temporary Telescoping Shaft Casing

Where the acceleration coefficient used for seismic design of the structure, as specified in the General Notes of the Structure Plans, is less than or equal to 0.16, the Contractor may use temporary telescoping casing for the shafts at bridge intermediate or interior piers, subject to the following conditions:

1. The Contractor shall submit the request to use temporary telescoping casing as a Type 2 Working Drawing. The request shall specify the diameters of the temporary telescoping casing, and shall specify the shafts where use is requested. The Contractor shall not proceed with the use of temporary telescoping casing until receiving the Engineer's approval.
2. The minimum diameter of the shaft shall be as shown in the Plans.
3. The temporary telescoping casing shall conform to Sections 6-19.3(3)B1, 6-19.3(3)B3, and 9-36.1(2).

The Contractor may use temporary telescoping casing for the shafts of bridge end piers, regardless of the acceleration coefficient used for the seismic design of the structure, subject to conditions 2 and 3 specified above and the following two additional conditions:

4. A maximum of two telescoping casing diameter changes will be allowed.
5. The maximum diameter change at each casing diameter transition shall be 12 inches.

6-19.3(3)B5 Permanent Slip Casing

Permanent slip casing is defined as casing installed vertically inside the temporary casing within the limits of the column-shaft splice zone, and wet-set into the shaft concrete no more than 3 feet below the shaft construction joint at the base of the shaft column splice zone, allowing subsequent removal of the temporary casing. The casing diameter requirements of Section 6-19.3(3)B1 do not apply to permanent slip casing, but the inside diameter of the permanent slip casing shall provide the steel reinforcing bar clearance specified in Section 6-19.3(5)C.

6-19.3(3)C Conduct of Shaft Casing Installation and Removal and Shaft Excavation Operations

The Contractor shall conduct casing installation and removal operations and shaft excavation operations such that the adjacent soil outside the casing and shaft excavation for the full height of the shaft is not disturbed. Disturbed soil is defined as soil whose geotechnical properties have been changed from those of the original in situ soil, and whose altered condition adversely affects the structural integrity of the shaft foundation. In no case shall shaft excavation and casing placement extend below the bottom of shaft excavation as shown in the Plans.

6-19.3(3)D Bottom of Shaft Excavation

The Contractor shall use appropriate means such as a cleanout bucket or air lift to clean the bottom of the excavation of all shafts. No more than 2 inches of loose or disturbed material shall be present at the bottom of the shaft just prior to placing concrete.

The excavated shaft shall be inspected and accepted by the Engineer prior to proceeding with construction. The bottom of the excavated shaft shall be sounded with an airlift pipe, a tape with a heavy weight attached to the end of the tape, or other means acceptable to the Engineer to determine that the shaft bottom meets the requirements in the Contract.

6-19.3(3)E Shaft Obstructions

When obstructions are encountered, the Contractor shall notify the Engineer promptly. An obstruction is defined as a specific object (including, but not limited to, boulders, logs, and previously fabricated objects) encountered during the shaft excavation operation which prevents or hinders the advance of the shaft excavation. When efforts to advance past the obstruction to the design shaft tip elevation result in the rate of advance of the shaft drilling equipment being significantly reduced relative to the rate of advance for the portion of the shaft excavation in the geological unit that contains the obstruction, then the Contractor shall remove, break up, or push aside the obstruction under the provisions of Section 6-19.5. The method of dealing with such obstructions, and the continuation of excavation shall be as proposed by the Contractor and accepted by the Engineer.

6-19.3(3)F Voids Between Permanent Casing and Shaft Excavation

When permanent casing is specified, excavation shall conform to the specified outside diameter of the shaft. After the casing has been filled with concrete, all void space occurring between the casing and shaft excavation shall be filled with a material that approximates the geotechnical properties of the in situ soils, in accordance with the shaft installation narrative specified in Section 6-19.3(2)B, item 5.

6-19.3(3)G Operating Shaft Excavation Equipment From an Existing Bridge

Drilling equipment shall not be operated from an existing bridge, except as otherwise noted. If necessary and safe to do so, and if the Contractor submits a Type 2 Working Drawing consisting of a written request in accordance with Section 6-01.6, the Engineer may permit operation of drilling equipment on a bridge.

6-19.3(3)H Seals for Shaft Excavation in Water

When shafts are constructed in water and the Plans show a seal between the casing shoring and the upper portion of the permanent casing of the shaft, the Contractor shall construct a seal in accordance with the shaft installation narrative specified in Section 6-19.3(2)B, item 7.

Concrete for the casing shoring seal shall be Class 4000W conforming to Section 6-02.

The seal thickness shown in the Plans is designed to resist the hydrostatic uplift force with the corresponding seal weight and adhesion of the seal to the permanent casing and the casing shoring of 20 psi, based on the casing shoring dimension and the seal vent water surface elevation specified in the Plans. If the Contractor uses a casing shoring diameter other than that specified in the Plans, the Contractor shall submit a revised seal design in accordance with Section 6-19.3(2)B, item 7.

6-19.3(3)I Required Use of Slurry in Shaft Excavation

6-19.3(3)I1 Uncased Shafts or Excavating Below Partial Depth Casing

The Contractor shall use slurry, in accordance with Section 6-19.3(4), to maintain a stable excavation during excavation and concrete placement operations whenever the shaft excavation extends below the highest ground water level indicated in the Plans, boring logs, or Summary of Geotechnical Conditions. If water is encountered, or expected to be encountered, at an elevation higher than that indicated by the Contract documents, then the Contractor shall use that elevation when using slurry in accordance with Section 6-19.3(4). If perched water tables are penetrated and sealed by temporary casing, slurry levels may be maintained below the perched water elevation provided the excavation remains stable and the slurry level is maintained above all unsealed water sources in accordance with 6-19.3(4)B.

6-19.3(3)I2 Excavation Within Temporary or Permanent Casing

The Contractor shall use slurry, in accordance with Section 6-19.3(4), to maintain a stable excavation during excavation and concrete placement operations once water begins to enter the shaft excavation at an infiltration rate of 12 inches of depth or more in 1 hour. If concrete is to be placed in the dry, the Contractor shall pump all accumulated water in the shaft excavation down to a 3-inch maximum depth prior to beginning concrete placement operations.

6-19.3(3)J Permanent Shaft Casing for Concrete-Filled Steel Tube (CFST) Shafts

CFST shafts are concrete shafts with permanent steel casing designed to function as a composite structural shaft. The soil within CFST shafts is removed either to the permanent casing tip or to the elevation shown in the Plans. The excavated void within CFST shafts is filled either with unreinforced structural concrete or reinforced structural concrete.

6-19.3(3)J1 Common Requirements for Shop Welding and Field Welding

Welded splices shall conform to Section 6-05.3(6) and as specified herein. A complete penetration groove weld between welded edges is required.

Welding for CFST shaft permanent casing shall conform to AWS D1.1/D1.1M, latest edition, Structural Welding Code, and Section 6-03.3(25), except that all weld filler metal shall be low hydrogen material selected from Table 4.1 in AASHTO/AWS D1.5M/D1.5, latest edition, Bridge Welding Code.

Welding and joint geometry for CFST shaft permanent casing seams, whether it be longitudinal, helical, or girth, shall be qualified in accordance with Clause 4, Qualification, of the AWS D1.1/D1.1M, latest edition, Structural Welding Code.

In addition, charpy V-notch (CVN) testing in accordance with Clause 4, Part D, of the AWS D1.1/D1.1M, latest edition, Structural Welding Code, shall be performed. CVN testing shall include five tests at 0°F. The acceptance threshold for the five samples shall meet an average value of 20-foot-pounds CVN for the set of test coupons and a minimum value of 15-foot-pounds CVN for all individual test coupons. The Contractor may submit documentation of prior qualification to the Engineer to satisfy this requirement.

The Contractor shall submit a Type 2 Working Drawing consisting of the weld procedure (WPS) and associated qualification testing. For ASTM A252 material, mill certification for each lot of steel to be welded shall accompany the Working Drawing submittal.

Skelp splices in spiral welded (helical seam) casing shall not be located within 12 inches of a girth shop or field weld.

Weld repairs shall conform to Section 5.25 of the AWS D1.1/D1.1M, latest edition, Structural Welding Code, using approved repair and weld procedures.

6-19.3(3)J2 Fabricating CFST Shaft Casing

The Contractor shall provide a minimum 14-day notice to the Engineer prior to the start of all CFST shaft permanent casing shop fabrication welding. Quality control for shop fabrication welding shall be conducted by an AWS Certified Welding Inspector (CWI).

Dimensional tolerances for CFST shaft permanent casing shall conform to the following requirements:

1. Out-of-roundness shall be within 1-percent of the nominal outside diameter.
2. Deviation from a straight line, parallel to the centerline of the pile, shall not exceed 0.001 times the length of the casing.
3. The maximum radial offset of the strip/plate edges shall be ⅛-inch. The offset shall be transitioned with a taper weld and the slope shall not be less than a 1 in 2.5 taper.
4. The bead height of weld reinforcement shall not exceed 3⁄16-inch.
5. Misalignment of weld beads for double-sided welded casing shall not exceed ⅛-inch.
6. The wall thickness shall not be less than 95-percent nor greater than 110-percent of the specified nominal thickness.

All seams, girth and skelp splices shall be complete penetration welds. All girth and skelp splices shall be 100 percent radiographically or ultrasonically inspected in accordance with either API 5L Annex E Section E.4 or E.5, or Table 6.2 and Clause 6 Part E, F or G in AWS D1.1/D1.1M, latest edition, Structural Welding Code. Additionally, 10-percent of the total length of seam welds for both longitudinal and helical welded casing, and one casing diameter length of seam centered on all skelp splice intersections, shall be randomly inspected as specified above. If repairs are required in more than 10-percent of the welds examined, additional inspection shall be performed. The additional inspection shall be made on both sides of the repair for a length equal to 10-percent of the length of the casing outside circumference. If repairs are required in more than 10-percent of welds examined in the second sample, 100-percent of the entire seam on the casing shall be inspected.

All seams, girth and skelp splices shall be 100 percent visually inspected in accordance with the acceptance criteria for statically loaded non-tubular connections in Table 6.1 of the AWS D1.1/D1.1M, latest edition, Structural Welding Code.

6-19.3(3)J3 Field Welding CFST Shaft Casing

The Contractor may perform field welding of CFST shaft permanent casing, provided a Type 2 Working Drawing of a field welding plan has been submitted to and accepted by the Engineer. The field welding plan shall include, at a minimum, the following:

1. Justification and description of need for the field welding operation.
2. Materials and equipment used to conduct the field welding operation.
3. Field welding procedures.

Ends of CFST shaft permanent casing shall be prepared for splicing in accordance with AWS D1.1/D1.1M, latest edition, Structural Welding Code. The ends shall also meet the fit-up requirements of AWS D1.1/D1.1M, latest edition, Structural Welding Code Section 9.24.1 Girth Weld Alignment (Tubular).

All splices shall be complete penetration groove welds using continuous backing rings of ¼ inch minimum thickness. Tack welds shall be located in the root of the complete penetration groove weld.

Field splice welds and welders shall be further qualified, tested and inspected as follows:

1. Welder qualification shall be performed on sample full girth sections of CFST shaft permanent casing to be used, or on a section of casing of the same diameter and with a minimum outside perimeter length of 2-feet, in the same position and using the same weld joint as for production CFST shaft permanent casing splicing.
2. Weld qualification tests shall be conducted in the presence of the Contractor's CWI and a representative of the Contracting Agency.
3. Unless otherwise specified in the Plans, field splices shall be 100-percent visually and ultrasonically inspected (UT) in accordance with the acceptance criteria for statically loaded non-tubular connections in Table 6.1 and the acceptance criteria in Table 6.2 in AWS D1.1/D1.1M, latest edition, Structural Welding Code.

The Contractor may submit a Type 1 Working Drawing of prior welder qualification performed on a CFST of the same diameter or smaller within the last six-months as a substitute for the above welder field weld qualification testing.

Quality control for field welding shall be conducted by an AWS CWI. The Contractor shall not begin splicing operations until receiving the CWI's approval of the joint fit-up. The CWI shall inspect 100 percent of all field welds in accordance with the criteria and requirements specified above. All field splices shall have received the CWI's approval prior to Engineer acceptance.

The CWI shall prepare a Type 1 Working Drawing documenting the results of the nondestructive quality control inspection of all field welds, and shall submit the report to the Engineer within five working days of the completion of the final splice in the project or as otherwise requested by the Engineer.

6-19.3(3)J4 Installing CFST Shaft Casing

Permanent casing for CFST shafts shall be open-ended, installed during shaft excavation operations to the depth shown in the Plans. Excavation in advance of the CFST permanent casing tip shall be limited to that specified in the accepted Shaft Installation Narrative Working Drawing and the limitations specified in the Plans, except in no case below the shaft tip elevation shown in the Plans.

6-19.3(3)J5 Interior Cleaning of CFST Shaft Casing

Prior to placing steel reinforcing bars and concrete, the interior surfaces of the CFST shaft permanent casing shall be cleaned. The cleaning methods and procedure shall be as described in the Shaft Construction Narrative Working Drawing submitted in accordance with Section 6-19.3(2). The cleaning shall remove native soils from the exposed interior surfaces of the CFST shaft permanent casing.

6-19.3(4) Slurry Installation Requirements

6-19.3(4)A Slurry Technical Assistance

If slurry other than water slurry is used, the manufacturer's representative, as identified to the Engineer in accordance with Section 6-19.3(2)C, shall:

1. Provide technical assistance for the use of the slurry,
2. Be at the site prior to introduction of the slurry into the first drilled hole requiring slurry, and
3. Remain at the site during the construction of at least the first shaft excavated to adjust the slurry mix to the specific site conditions.

After the manufacturer's representative is no longer present at the site, the Contractor's employee trained in the use of the slurry, as identified to the Engineer in accordance with Section 6-19.3(2)C, shall be present at the site throughout the remainder of shaft slurry operations for this project to perform the duties specified in items 1 through 3 above.

6-19.3(4)B Minimum Level of Slurry in the Excavation

When slurry is used in a shaft excavation the following is required:

1. The height of the slurry shall be as required to provide and maintain a stable hole to prevent bottom heave, caving, or sloughing of all unstable zones.
2. The Contractor shall provide casing, or other means, as necessary to meet these requirements.
3. The slurry level in the shaft while excavating shall be maintained above the groundwater level the greater of the following dimensions:
    a. Not less than 5 feet for mineral slurries.
    b. Not less than 10 feet for water slurries.
    c. Not less than 10 feet for synthetic slurries.
4. The slurry level in the shaft throughout all stops as specified in Section 6-19.3(3)A and during concrete placement as specified in Section 6-19.3(7) shall be no lower than the water level elevation outside the shaft.

6-19.3(4)C Slurry Sampling and Testing

Mineral slurry and synthetic slurry shall be mixed and thoroughly hydrated in slurry tanks, ponds, or storage areas. The Contractor shall draw sample sets from the slurry storage facility and test the samples for conformance with the specified viscosity and pH properties before beginning slurry placement in the drilled hole. Mineral slurry shall conform to the material specifications in Section 9-36.2(1). Synthetic slurry shall conform to Section 9-36.2(2), the quality control plan included in the shaft installation narrative in accordance with Section 6-19.3(2)B, item 4. A sample set shall be composed of samples taken at mid-height and within 2 feet of the bottom of the storage area.

When synthetic slurry is used, the Contractor shall keep a written record of all additives and concentrations of the additives in the synthetic slurry. These records shall be submitted as a Type 1 Working Drawing once the slurry system has been established in the first drilled shaft on the project. The Contractor shall provide revised data to the Engineer if changes are made to the type or concentration of additives during construction.

The Contractor shall sample and test all slurry in the presence of the Engineer, unless otherwise directed. The date, time, names of the persons sampling and testing the slurry, and the results of the tests shall be recorded. A copy of the recorded slurry test results shall be submitted to the Engineer at the completion of each shaft, and during construction of each shaft when requested by the Engineer.

Sample sets of all slurry, composed of samples taken at mid-height and within 2 feet of the bottom of the shaft and the storage area, shall be taken and tested once every 4 hours minimum at the beginning and during drilling shifts and prior to cleaning the bottom of the hole to verify the control of the viscosity and pH properties of the slurry. Sample sets of all slurry shall be taken and tested at least once every 2 hours if the previous sample set did not have consistent viscosity and pH properties. All slurry shall be recirculated, or agitated with the drilling equipment, when tests show that the sample sets do not have consistent viscosity and pH properties. Cleaning of the bottom of the hole shall not begin until tests show that the samples taken at mid-height and within 2 feet of the bottom of the hole have consistent viscosity and pH properties.

Sample sets of all slurry, as specified, shall be taken and tested to verify control of the viscosity, pH, density, and sand content properties after final cleaning of the bottom of the hole just prior to placing concrete. Placement of the concrete shall not start until tests show that the samples taken at mid-height and within 2 feet of the bottom of the hole have consistent specified properties.

6-19.3(4)D Maintenance of Required Slurry Properties

The Contractor shall clean, recirculate, de-sand, or replace the slurry to maintain the required slurry properties.

6-19.3(4)E Maintenance of a Stable Shaft Excavation

The Contractor shall demonstrate to the satisfaction of the Engineer that stable conditions are being maintained. If the Engineer determines that stable conditions are not being maintained, the Contractor shall immediately take action to stabilize the shaft. The Contractor shall submit a revised shaft installation narrative that addresses the problem and prevents future instability. The Contractor shall not continue with shaft construction until the damage that has already occurred is repaired in accordance with the specifications, and until receiving the Engineer's review of the revised shaft installation narrative.

When mineral slurry conforming to Section 9-36.2(1) is used to stabilize the unfilled portion of the shaft, the Contractor shall remove the excess slurry buildup inside of the shaft diameter prior to continuing with concrete placement. The Contractor shall use the same methods of shaft excavation and the same diameter of drill tools to remove the excess slurry buildup as was used to excavate the shaft to its current depth.

6-19.3(4)F Disposal of Slurry and Slurry Contacted Spoils

The Contractor shall manage and dispose of the slurry wastewater in accordance with Section 8-01.3(1)C. Slurry-contacted spoils shall be disposed of as specified in the shaft installation narrative in accordance with Section 6-19.3(2)B, item 8, and in accordance with the following requirements:

1. Uncontaminated spoils in contact with water-only slurry may be disposed of as clean fill.
2. Uncontaminated spoils in contact with water slurry mixed with flocculants approved in Section 8-01.3(1)C3 may be disposed of as clean fill away from areas that drain to surface waters of the state.
3. Spoils in contact with synthetic slurry or water slurry with polymer-based additives or flocculants not approved in Section 8-01.3(1)C3 shall be disposed of in accordance with Section 3-03.3(7)C. With permission of the Engineer, the Contractor may re-use these spoils on-site.
4. Spoils in contact with mineral slurry shall be disposed of in accordance with Section 3-03.3(7)C. With permission of the Engineer, the Contractor may re-use these spoils on-site.

6-19.3(5) Assembly and Placement of Reinforcing Steel

6-19.3(5)A Steel Reinforcing Bar Cage Assembly

The reinforcing cage shall be rigidly braced to retain its configuration during handling and construction. Individual or loose bars will not be permitted. The Contractor shall show bracing and all extra reinforcing steel required for fabrication of the cage on the shop drawings. Shaft reinforcing bar cages shall be supported on a continuous surface to the extent possible. All rigging connections shall be located at primary handling bars, as identified in the reinforcing steel assembly and installation plan. Internal bracing is required at each support and lift point.

The reinforcement shall be carefully positioned and securely fastened to provide the minimum clearances listed below, and to ensure no displacement of the reinforcing steel bars occurs during placement of the concrete. The steel reinforcing bars shall be securely held in position throughout the concrete placement operation.

6-19.3(5)B Steel Reinforcing Bar Cage Centralizers

The Contractor shall submit details of the proposed reinforcing cage centralizers along with the shop drawings. The reinforcing steel centralizers at each longitudinal space plane shall be placed at least at the quarter points around the circumference of the steel reinforcing bar cage, and at a maximum longitudinal spacing of either 2.5 times the shaft diameter or 20 feet, whichever is less. The Contractor shall furnish and install additional centralizers as required to maintain the specified concrete cover throughout the length of the shaft.

6-19.3(5)C Concrete Cover Over Steel Reinforcing Bars

Steel reinforcing bars shall be placed as shown in the Plans with minimum concrete cover as shown below:

Shaft Diameter (feet)	Minimum Concrete Cover, and Concrete Cover Tolerance, Except at Permanent Slip Casing (inches)	Minimum Concrete Cover at Permanent Slip Casing (inches)
Less than or equal to 3	3, -1½	1½
Greater than 3 and less than 4	4, -2	1½
Greater than or equal to 4 and less than 5	4, -2	2
5 or larger	6, -3	3

The concrete cover tolerances specified above apply to the concrete cover specified in the Plans, even if it exceeds the minimum concrete cover.

6-19.3(5)D Steel Reinforcing Bar Cage Support at Base of Shaft Excavation

For shafts with temporary casing within 15 feet of the bottom of shaft elevation as specified in the Plans, the Contractor may place quarry spalls or other rock backfill acceptable to the Engineer into the shaft below the specified bottom of shaft elevation as a means to support the steel reinforcing bar cage, provided that the materials and means to accomplish this have been addressed by the shaft installation narrative, as specified in Section 6-19.3(2)B, item 9. The use of bar boots and/or rebar cage base plates is required when quarry spalls or other rock backfill is placed at the base of the shaft excavation.

6-19.3(6) Contractor Furnished Accessories for Nondestructive QA Testing

6-19.3(6)A Shafts Requiring Access Tubes

The Contractor shall furnish and install access tubes in all shafts receiving CSL testing or the thermal probe method of TIP testing, except as otherwise noted in Section 6-19.3(1)B1.

6-19.3(6)B Orientation and Assembly of the Access Tubes

The Contractor shall securely attach the access tubes to the interior of the reinforcement cage of the shaft. One access tube shall be furnished and installed for each foot of shaft diameter, rounded to the nearest whole number, as shown in the Plans. The number of access tubes for shaft diameters specified as “X feet 6 inches” shall be rounded up to the next higher whole number. The access tubes shall be placed around the shaft, inside the spiral or hoop reinforcement, and bundled with the vertical reinforcement. Where circumferential components of the rebar cage bracing system prevent bundling the access tubes directly to the vertical reinforcement, the access tubes shall be placed inside the circumferential components of the rebar cage bracing system as close as possible to the nearest vertical steel reinforcement bar.

The access tubes shall be installed in straight alignment and as near to parallel to the vertical axis of the reinforcement cage as possible. The access tubes shall extend from the bottom of the reinforcement cage to at least 2 feet above the top of the shaft. Splice joints in the access tubes, if required to achieve full length access tubes, shall be watertight. The Contractor shall clear the access tubes of all debris and extraneous materials before installing the access tubes. The tops of access tubes shall be deburred. Care shall be taken to prevent damaging the access tubes during reinforcement cage installation and concrete placement operations in the shaft excavation.

6-19.3(6)C Care for Access Tubes From Erection Through Nondestructive QA Testing

The access tubes shall be filled with potable water before concrete placement, and the top watertight PVC caps shall be reinstalled and secured in accordance with Section 9-36.4. The Contractor shall keep all of a shaft’s access tubes full of water through the completion of nondestructive QA testing of that shaft. When temperatures below freezing are possible, the Contractor shall protect the access tubes against freezing by wrapping the exposed tubes with insulating material, adding antifreeze to the water in the tubes, or other methods acceptable to the Engineer.

6-19.3(6)D Shafts Requiring Thermal Wire

The Contractor shall furnish and install thermal wire in all shafts receiving the thermal wire method of TIP testing, except as otherwise noted in Section 6-19.3(1)B1.

6-19.3(6)E Thermal Wire and Thermal Access Points (TAPs)

The thermal wire and associated couplers shall be obtained from the source specified in the Special Provisions.

The Contractor shall securely attach the thermal wire to the interior of the reinforcement cage of the shaft in conformance with the supplier’s instructions. At a minimum, one thermal wire shall be furnished and installed for each foot of shaft diameter, rounded to the nearest whole number, as shown in the Plans. The number of thermal wires for shaft diameters specified as “X feet 6 inches” shall be rounded up to the next higher whole number. The thermal wires shall be placed around the shaft, inside the spiral or hoop reinforcement, and tied to the vertical reinforcement with plastic “zip” ties at a maximum spacing of 2-feet. Steel tie wire shall not be used.

The thermal wire shall be installed in straight alignment and taut, but with enough slack to not be damaged during reinforcing cage lofting. The wires shall be as near to parallel to the vertical axis of the reinforcement cage as possible. The thermal wire shall extend from the bottom of the reinforcement cage to the top of the shaft, with a minimum of 5-feet of slack wire provided above the top of shaft. All thermal wires in a shaft shall be equal lengths. Care shall be taken to prevent damaging the thermal wires during reinforcement cage installation and concrete placement operations in the shaft excavation.

After completing shaft reinforcement cage fabrication at the site and prior to installation of the cage into the shaft excavation, the Contractor shall install and connect thermal access points (TAPs) to the thermal wires. The TAPs shall record data for at least one hour after the cage is placed in the excavation to measure the slurry temperature and enable the steel and slurry temperatures to equilibrate prior to placing concrete in the shaft. The TAPs shall record and store data every 15 minutes. The TAPs shall remain active for a minimum of 36 hours.

Prior to beginning concrete placement the TAPs shall be checked to ensure they are recording data and that the wires have not been damaged. If a TAP unit is not functioning due to a damaged wire, the Contractor shall repair or replace the wire. If a TAP unit fails or a wire breaks after concrete placement has started, the Contractor shall not stop the concrete placement operation to repair the wire.

6-19.3(6)F Use of Access Tubes for TIP Testing Under the Thermal Probe Method

The Contractor may use access tubes for TIP testing under the thermal probe method. Access tubes shall be cared for in accordance with Section 6-19.3(6)C. Prior to TIP testing under the thermal probe method, the water in each tube shall be removed, collected, and stored in an insulated container. The access tube shall be blown dry and swabbed to remove residual water. After TIP testing, the collected and stored tube water shall be introduced back into the access tube. New potable water may be used, provided the water temperature is not more than 10°F cooler than the average concrete temperature measured by the probe.

6-19.3(7) Placing Concrete

6-19.3(7)A Concrete Class for Shaft Concrete

Shaft concrete shall be Class 5000P conforming to Section 6-02.

6-19.3(7)B Concrete Placement Requirements

Concrete placement shall commence immediately after completion of excavation by the Contractor and inspection by the Engineer. Immediately prior to commencing concrete placement, the shaft excavation and the properties of the slurry (if used) shall conform to Sections 6-19.3(3)D and 6-19.3(4), respectively. Concrete placement shall continue in one operation to the top of the shaft, or as shown in the Plans.

During concrete placement, the Contractor shall monitor, and minimize, the difference in the level of concrete inside and outside of the steel reinforcing bar cage. The Contractor shall conduct concrete placement operations to maintain the differential concrete head as 1-foot maximum.

If water is not present, the concrete shall be deposited through the center of the reinforcement cage by a method that prevents segregation of aggregates and splashing of concrete on the reinforcement cage. The concrete shall be placed such that the free-fall is vertical down the center of the shaft without hitting the sides, the steel reinforcing bars, or the steel reinforcing bar cage bracing. The Section 6-02.3(6) restriction for 5 feet maximum free fall shall not apply to placement of concrete into a shaft.

6-19.3(7)C Concrete Vibration Requirements

When placing concrete in the dry, only the top 5 feet of concrete shall be vibrated, in accordance with Section 6-02.3(9), except that the entire depth of concrete placed in the shaft-column steel reinforcing bar splice zone shall be vibrated. If a temporary casing is used, it shall be removed before vibration. This requirement may be waived if a temporary casing is used and removed with a vibratory hammer during the concrete placement operation. Vibration of concrete does not affect the maximum slump allowed for the concrete class specified.

6-19.3(7)D Requirements for Placing Concrete Underwater

When placing concrete underwater, including when water in a shaft excavation exceeds 3 inches in depth, the Contractor shall place the concrete by pressure feed using a concrete pump, with a watertight tube having a minimum diameter of 4 inches. The discharge end of the tube on the concrete pump shall include a device to seal out water while the tube is first filled with concrete. Alternatively, the Contractor may use a plug that is inserted at the hopper of the concrete pump and travels through the tremie to keep the concrete separated from the water and slurry. Concrete placement by gravity feed is not allowed.

Throughout the underwater concrete placement operation, the discharge end of the tube shall remain submerged in the concrete at least 5 feet and the tube shall always contain enough concrete to prevent water from entering. The Contractor shall remove all liquid from above the shaft construction joint prior to removing the tremie. The concrete placement shall be continuous until the work is completed, resulting in a seamless, uniform shaft.

6-19.3(7)E Testing and Repair of Shaft Concrete Placed Underwater

If the underwater concrete placement operation is interrupted, the Engineer may require the Contractor to prove by core drilling or other tests that the shaft contains no voids or horizontal joints. If testing reveals voids or joints, the Contractor shall repair them or replace the shaft at no expense to the Contracting Agency. Responsibility for coring costs, and calculation of time extension, shall be in accordance with Section 6-19.3(9)H.

6-19.3(7)F Shaft Construction Joint

Before placing fresh concrete above the shaft construction joint, the Contractor shall remove all scum, laitance, loose gravel, weak or unsound concrete and sediment from the top of the construction joint. The Contractor shall chip off high spots on the surface of the shaft concrete that would prevent the steel reinforcing bar cage from being placed in the position required in the Plans.

Crosshole sonic log testing operations specified in Section 6-19.3(9) may be performed prior to preparing the shaft construction joint as specified herein.

6-19.3(7)G Protection of Fresh and Curing Concrete From Vibration

The Contractor’s construction operation in the vicinity of a shaft excavation with freshly placed concrete and curing concrete shall conform to Section 6-02.3(6)D.

6-19.3(7)H Uniform Yield Form

Except for shafts where the shaft concrete is placed in the dry, the Contractor shall complete a uniform yield form, consistent with the sample form submitted to the Engineer as part of the shaft installation narrative as specified in Section 6-19.3(2)B, item 6, for each shaft and shall submit the completed form to the Engineer within 24 hours of completing the concrete placement in the shaft.

6-19.3(7)I Requirements for Placing Concrete Above the Top of Shaft

Concrete shall not be placed above the top of shaft (for column splice zones, columns, footings, or shaft caps) until the Contractor receives the Engineer's acceptance of nondestructive QA testing, if performed at that shaft, and acceptance of the shaft.

6-19.3(8) Casing Removal

6-19.3(8)A Concrete Head Requirements During Temporary Casing Removal

As the temporary casing is withdrawn, the Contractor shall maintain the concrete and slurry inside the casing at a level sufficient to balance the hydrostatic pressure outside the casing.

6-19.3(8)B Removing Portions of Permanent Casing Above the Top of Shaft

Tops of permanent casings for the shafts shall be removed to the top of the shaft or finished groundline, whichever is lower, unless directed otherwise by the Engineer. For those shafts constructed within a permanent body of water, tops of permanent casings for shafts shall be removed to the low water elevation, unless directed otherwise by the Engineer.

6-19.3(8)C Requirements for Leaving Temporary Casing in Place

The Contractor shall completely remove all temporary casings, except as noted. The Contractor may leave some or all of the temporary casing in place provided all the following conditions are satisfied:

1. The Contractor shall submit a Type 2E Working Drawing of the following information:
    a. The Contractor shall completely describe the portion of the temporary casing to remain.
    b. The Contractor shall specify the reason(s) for leaving the portion of the temporary casing in place.
    c. The Contractor shall submit structural calculations, using the design specifications and design criteria specified in the General Notes of the structure Plans, indicating that leaving the temporary casing in place is compatible with the structure as designed in the Plans.

6-19.3(9) Nondestructive QA Testing of Shafts

The Contractor shall provide nondestructive QA testing and analysis on all shafts with access tubes or thermal wires and TAPs facilitating the testing (See Section 6-19.3(1)B). The testing and analysis shall be performed by the testing organizations identified by the Contractor's submittal in accordance with Section 6-19.3(2)D.

The Engineer may direct that additional testing be performed at a shaft if anomalies or a soft bottom are detected by the Contractor's testing. If additional testing at a shaft confirms the presence of a defect(s) in the shaft, the testing costs and the delay costs resulting from the additional testing shall be borne by the Contractor in accordance with Section 1-05.6. If the additional testing indicates that the shaft has no defect, the testing costs and the delay costs resulting from the additional testing will be paid by the Contracting Agency in accordance with Section 1-05.6, and, if the shaft construction is on the critical path of the Contractor's schedule, a time extension equal to the delay created by the additional testing will be granted in accordance with Section 1-08.8.

6-19.3(9)A TIP Testing Using Thermal Probes or CSL Testing

If selected as the nondestructive QA testing method by the Contractor, TIP testing using thermal probes, or CSL testing shall be performed after the shaft concrete has cured at least 96 hours. Additional curing time prior to testing may be required if the shaft concrete contains admixtures, such as set retarding admixture or water-reducing admixture, added in accordance with Section 6-02.3(3). The additional curing time prior to testing required under these circumstances shall not be grounds for additional compensation or extension of time to the Contractor in accordance with Section 1-08.8.

6-19.3(9)B Inspection of Access Tubes

After placing the shaft concrete and before beginning the crosshole sonic log testing of a shaft, the Contractor shall inspect the access tubes. Each access tube that the test probe cannot pass through shall be replaced, at the Contractor's expense, with a 2-inch diameter hole cored through the concrete for the entire length of the shaft. Unless directed otherwise by the Engineer, cored holes shall be located approximately 6 inches inside the reinforcement and shall not damage the shaft reinforcement. The Contractor shall submit a Type 2 Working Drawing describing the conduct of the core hole drilling operation including measures to ensure core hole verticality and avoidance of reinforcement. Descriptions of inclusions and voids in cored holes shall be logged and a copy of the log shall be submitted to the Engineer. Findings from cored holes shall be preserved, identified as to location, and made available for inspection by the Engineer.

6-19.3(9)C TIP Testing With Thermal Wires and TAPs

If selected as the nondestructive QA testing method by the Contractor, TIP testing with thermal wires and TAPs (See Section 6-19.3(6)E) shall be performed. The TIP testing shall commence at the beginning of the concrete placement operation, recording temperature readings at 15-minute intervals until the peak temperature is captured in the data. Additional curing time may be required if the shaft concrete contains admixtures, such as set retarding admixture or water-reducing admixture, added in accordance with Section 6-02.3(3). The additional curing time required under these circumstances shall not be grounds for additional compensation or extension of time to the Contractor in accordance with Section 1-08.8.

TIP testing shall be conducted at all shafts in which thermal wires and TAPs have been installed for thermal wire analysis (Section 6-19.3(6)A).

6-19.3(9)D Nondestructive QA Testing Results Submittal

The Contractor shall submit the results and analysis of the nondestructive QA testing for each shaft tested. The Contractor shall submit the test results within three working days of testing. Results shall be a Type 2E Working Drawing presented in a written report.

TIP reports shall include:

1. A map or plot of the wire/tube location within the shaft and their position relative to a known and identifiable location, such as North.
2. Graphical displays of temperature measurements versus depth of each wire or tube for the analysis time selected, overall average temperature with depth, shaft radius or diameter with depth, concrete cover versus cage position with depth, and effective radius.
3. The report shall identify unusual temperatures, particularly significantly cooler local deviations from the overall average.
4. The report shall identify the location and extent where satisfactory or questionable concrete is identified.
    a. Satisfactory (S) – 0 to 6 percent Effective Radius Reduction and Cover Criteria Met
    b. Questionable (Q) – Effective Local Radius Reduction > 6 percent, Effective Local Average Diameter Reduction > 4 percent, or Cover Criteria Not Met

5. Variations in temperature between wire/tubes (at each depth) which in turn correspond to variations in cage alignment.
6. Where shaft specific construction information is available (e.g. elevations of the top of shaft, bottom of casing, bottom of shaft), these values shall be noted on all pertinent graphical displays.

CSL reports shall include:

1. A map or plot of the tube location within the shaft and their position relative to a known and identifiable location, such as North.
2. Graphical displays of CSL Energy versus Depth and CSL signal arrival time versus depth or velocity versus depth.
3. The report shall identify the location and extent where good, questionable, and poor concrete is identified, where no signal was received, or where water is present.
    a. Good (G) – No signal distortion and decrease in signal velocity of 10 percent or less is indicative of good quality concrete.
    b. Questionable (Q) – Minor signal distortion and a lower signal amplitude with a decrease in signal velocity between 10 percent and 20 percent.
    c. Poor (P) – Severe signal distortion and much lower signal amplitude with a decrease in signal velocity of 20 percent or more.
    d. No Signal (NS) – No signal was received.
    e. Water (W) – A measured signal velocity of nominally V = 4,800 to 5,000 fps.

All QA test reports will provide a recommendation to accept the shaft as-is, recommendation for further review by the Engineer, or will provide a plan for further testing, investigation or repair to address all deficiencies identified by the testing.

6-19.3(9)E Vacant

6-19.3(9)F Contractor's Investigation and Remedial Action Plan

For all shafts determined to be unacceptable, the Contractor shall submit a Type 2 Working Drawing consisting of a plan for further investigation or remedial action. All modifications to the dimensions of the shafts, as shown in the Plans, required by the investigation and remedial action plan shall be supported by calculations and working drawings. All investigation and remedial correction procedures and designs shall be submitted.

6-19.3(9)G Rejection of Shafts and Revisions to Concrete Placement Operations

If the Engineer determines that the concrete placed under slurry for a given shaft is structurally inadequate, that shaft will be rejected. The placement of concrete under slurry shall be suspended until the Contractor submits to the Engineer written changes to the methods of shaft construction needed to prevent future structurally inadequate shafts, and receives the Engineer's written approval of the submittal.

6-19.3(9)H Cored Holes

At the Engineer's request, the Contractor shall drill a corehole in locations with questionable shaft quality (as determined from crosshole sonic log testing and analysis or by observation of the Engineer) to explore the shaft condition.

Prior to beginning coring, the Contractor shall submit Type 2 Working Drawings consisting of the method and equipment used to drill and remove cores from shaft concrete. The coring method and equipment shall provide for complete core recovery and shall minimize abrasion and erosion of the core.

If a defect is confirmed, the Contractor shall pay for all coring costs in accordance with Section 1-05.6. If no defect is encountered, the Contracting Agency will pay for all coring costs in accordance with Section 1-05.6, and, if the shaft construction is on the critical path of the Contractor's schedule, compensation for the delay will be granted by an appropriate time extension in accordance with Section 1-08.8. Materials and Work necessary, including engineering analysis and redesign, to effect corrections for shaft defects shall be furnished to the Engineer's satisfaction at no additional cost to the Contracting Agency.

6-19.3(9)I Requirements for Access Tubes and Cored Holes After CSL Testing

All access tubes and cored holes shall be dewatered and filled with grout conforming to Section 9-36.5 after tests are completed. The access tubes and cored holes shall be filled using grout tubes that extend to the bottom of the tube or hole or into the grout already placed.

6-19.3(10) Engineer's Final Acceptance of Shafts

The Engineer will determine final acceptance of each shaft, based on the nondestructive QA test results and analysis for the tested shafts, and will provide a response to the Contractor within 3 working days after receiving the test results and analysis submittal.

6-19.4 Measurement

Constructing shafts will be measured by the linear foot. The linear foot measurement will be calculated using the top of shaft elevation and the bottom of shaft elevation for each shaft as shown in the Plans.

Rock excavation for shaft, including haul, will be measured by the linear foot of shaft excavated. The linear feet measurement will be computed using the top of the rock line, defined as the highest bedrock point within the shaft diameter, and the bottom elevation shown in the Plans.

QA shaft test will be measured once per shaft tested.

6-19.5 Payment

Payment will be made for the following Bid items when they are included in the Proposal:

"Constructing___Diam. Shaft", per linear foot.

The unit Contract price per linear foot for "Constructing___Diam. Shaft" shall be full pay for performing the Work as specified, including:

1. Soil excavation for shaft, including all costs in connection with furnishing, mixing, placing, maintaining, containing, collecting, and disposing of all mineral, synthetic and water slurry, and disposing of groundwater collected by the excavated shaft.
2. Furnishing and placing temporary shaft casing, including temporary casing in addition to the required casing specified in the Special Provisions, and including all costs in connection with completely removing the casing after completing shaft construction.
3. Furnishing permanent casing for shaft.
4. Placing permanent casing for shaft.
5. Casing shoring, including all costs in connection with furnishing and installing casing shoring above the specified upper limit for casing shoring but necessary to provide for sufficient water head pressure to resist artesian water pressure present in the shaft excavation, removing casing shoring, and placing seals when required.
6. Furnishing and placing steel reinforcing bar and epoxy-coated steel reinforcing bar, including furnishing and installing steel reinforcing bar centralizers.

7. Installation of CSL tubes or thermal wires.
8. Furnishing, placing and curing concrete to the top of shaft or to the construction joint at the base of the shaft-column splice zone as applicable.
9. Cleaning and preparing the shaft construction joint.

Payment for “Constructing___Diam. Shaft” will be made upon Engineer acceptance of the shaft, including completion of satisfactory QA shaft tests as applicable.

“Rock Excavation For Shaft Including Haul”, per linear foot.

When rock excavation is encountered, payment for rock excavation is in addition to the unit Contract price per linear foot for “Constructing___Diam. Shaft”

“Shoring Or Extra Excavation Cl. A - ___”, lump sum.

The lump sum Contract price for “Shoring Or Extra Excavation Cl. A - ___” shall be full pay for performing the Work as specified, including all costs in connection with all excavation outside the limits specified for soil and rock excavation for shaft including haul, all temporary telescoping casings, and all temporary casings beyond the limits of required temporary casing specified in the Special Provisions.

“QA Shaft Test”, per each.

The unit Contract price per each for “QA Shaft Test” shall be full pay for performing the Work as specified, including operating all associated accessories necessary to record and process data and develop the summary QA test reports. Section 1-04.6 does not apply to this bid item.

“Removing Shaft Obstructions”, estimated.

Payment for removing, breaking-up, or pushing aside shaft obstructions, as defined in Section 6-19.3(3)E, will be made for the changes in shaft construction methods necessary to deal with the obstruction. The Contractor and the Engineer shall evaluate the effort made and reach agreement on the equipment and employees utilized, and the number of hours involved for each. Once these cost items and their duration have been agreed upon, the payment amount will be determined using the rate and markup methods specified in Section 1-09.6. For the purpose of providing a common proposal for all Bidders, the Contracting Agency has entered an amount for the item “Removing Shaft Obstructions” in the Bid Proposal to become a part of the total Bid by the Contractor.

If drilled shaft tools, cutting teeth, casing or Kelly bar is damaged as a result of the obstruction removal work, the Contractor will be compensated for the costs to repair this equipment in accordance with Section 1-09.6.

If shaft construction equipment is idled as a result of the Work required to deal with the obstruction and cannot be reasonably reassigned within the project, then standby payment for the idled equipment will be added to the payment calculations. If labor is idled as a result of the Work required to deal with the obstruction and cannot be reasonably reassigned within the project, then all labor costs resulting from Contractor labor agreements and established Contractor policies will be added to the payment calculations.

The Contractor shall perform the amount of obstruction Work estimated by the Contracting Agency within the original time of the Contract. The Engineer will consider a time adjustment and additional compensation for costs related to the extended duration of the shaft construction operations, provided:

1. The dollar amount estimated by the Contracting Agency has been exceeded, and
2. The Contractor shows that the obstruction removal Work represents a delay to the completion of the project based on the current progress schedule provided in accordance with Section 1-08.3.

6-20 Buried Structures

6-20.1 Description

This Work consists of designing and constructing Buried Structures of the various types including associated approach slabs, footings, headwalls, wingwalls, Class 4000D concrete topping slab, connected barriers, rails, and fall protection in accordance with the Contract documents.

6-20.1(1) Definitions

Buried Structure: A Structure consisting of one of the types defined below:

> **Concrete Three Sided Structure:** A precast or cast-in-place reinforced concrete structure with vertical walls and an integral top slab placed on reinforced concrete foundations. It may be arched or corded and may contain chamfers or fillets.
>
> **Concrete Box:** A precast or cast-in-place reinforced concrete structure with vertical walls, an integral top slab and an integral bottom slab. The top and bottom slabs are usually flat and at 90-degrees to the walls forming a rectangular structure. The corners may contain chamfers or fillets.
>
> **Concrete Split Box:** A concrete box culvert split into upper and lower units by horizontal joints in the vertical walls.
>
> **Structural Plate Pipe:** A steel or aluminum structural plate around the entire circumference of a pipe shape. Structural plate pipes may contain external reinforcing members and multiple radiuses and plate thicknesses. Structural plate pipe shapes include but are not limited to round, ellipse, underpass, pipe-arch and pear.
>
> **Structural Plate Arch:** A steel or aluminum structural plate arch shape placed on reinforced concrete foundations. Structural plate arches may contain external reinforcing members and multiple radiuses and plate thicknesses.
>
> **Structural Plate Box:** A steel or aluminum structural plate box shape that meets the requirements of the AASHTO *LRFD Bridge Design Specifications* Section 12.9 (or Section 12.8.9 for deep corrugated structural plate structures with ratio of crown radius to haunch radius > 5).

Headwall: Structure elements employed as end treatments connected to or constructed on top of Buried Structures, including, at a minimum, parapets, slope collars, and cutoff walls.

Wingwall: A retaining wall structure element adjacent to or above a Buried Structure end or headwall.

Class 4000D Concrete Topping Slab: A 5.0-inch minimum thickness composite cast-in-place system with at least one mat of epoxy coated reinforcement.

Structure Free Zone (SFZ): An imaginary prism defined in the Contract documents which represents the minimum boundary within which no part of the buried structure including foundations, headwalls and wingwalls shall be allowed. For a Contracting Agency Supplied Design the SFZ shall be defined by the interior horizontal and vertical dimensions and the length of the structure shown in the Contract documents, unless noted otherwise.

Hydraulic Design Flood Elevation: 100-year or the 2080 projected 100-year flood elevation.

Zone of Influence: A prism defined in the Contract documents representing the boundary within which the Structure derives support from the surrounding granular backfill or native soil through soil-structure interaction.

6-20.2 Materials

Materials shall meet the requirements of the following sections:

Controlled Density Fill (CDF)	3-07.3(1)E
Lean Concrete	6-02.3(2)D
Cement	9-01
Aggregates for Concrete	9-03.1
AASHTO Grading No. 57	9-03.1(4)C
Aggregates for Ballast and Crushed Surfacing	9-03.9
Streambed Aggregates	9-03.11
Gravel Backfill	9-03.12
Borrow	9-03.14
Joint Sealing Materials	9-04
Structural Plate Pipes, Arches and Boxes	9-05.6
Deformed Steel Bars	9-07.2
Epoxy-Coated Steel Reinforcing Bars	9-07.3
Galvanizing Repair Paint, High Zinc Dust Content	9-08.1(2)B
Primer, Zinc-Rich, Single-Component, Moisture-Cured Polyurethane	9-08.1(2)F
NEPCOAT Qualified Business Products List B Primer	9-08.1(2)N
Grout Type 2 for Nonshrink Applications	9-20.3(2)
Mortar	9-20.4
Concrete Curing Materials and Admixtures	
Water	9-25

Concrete class shall be as specified but in no case shall be less than Class 4000.

Materials for Class 4000D concrete topping slab shall meet all the requirements of Class 4000D concrete as specified in Section 6-02.2.

6-20.3 Construction Requirements

6-20.3(1) Design

6-20.3(1)A Design Delivery Method

Buried Structures shall be considered a Contractor Supplied Design when the Contract documents do not include a complete set of design details for a Structure (consisting of, at a minimum, defining material requirements, shapes, dimensions, reinforcing details, joint and connection details, etc.).

Buried Structures shall be considered a Contracting Agency Supplied Design when the Contract documents include a complete set of Structural Engineer stamped design details for a Structure.

Contractor or Contracting Agency Supplied Designs, may include Standard Plans, as applicable.

6-20.3(1)A1 Contractor Supplied Design

The Contractor shall prepare the design in accordance with Sections 6-20.3(1)C through 6-20.3(1)I. All submittal requirements of Section 6-20.3(2) shall apply.

The Buried Structure, headwalls and wingwalls shall be located as specified in the Contract documents, including but not limited to the alignment, length, profile, and elevation. The Buried Structure shall accommodate the geometry required in the Contract documents. No part of the Buried Structure, including but not limited to foundations, headwalls and wingwalls, shall be allowed within the Structure Free Zone. The Buried Structure and wall types shall be as specified in the Contract documents.

6-20.3(1)A2 Contracting Agency Supplied Design

The Buried Structure design shall be as presented in the Contract documents. The requirements of Sections 6-20.3(1)C through 6-20.3(1)I, 6-20.3(2)A and 6-20.3(2)B shall not apply for the Structure. All other submittal requirements of Section 6-20.3(2) shall apply.

Any alternate structure proposals shall meet the requirements of Section 1-04.4(2).

The Contractor may substitute welded wire reinforcement (WWR) conforming to Section 9-07.7 for plain deformed bar. The WWR shall provide an equivalent bar area, equal or reduced bar spacing , and shall satisfy crack control and minimum reinforcement requirements of the AASHTO LRFD Bridge Design Specifications. The WWR shall be deformed, and the specified minimum yield strength shall be limited to a maximum of 75 ksi.

The proposed WWR substitution, with supporting calculations, shall be submitted to the Engineer as a Type 2E Working Drawing.

6-20.3(1)B Buried Structure Class

The Structural Clear Span of a Buried Structure shall be used to determine the Buried Structure class. When supporting a Roadway, the Structural Clear Span shall be the widest horizontal opening from interior face to interior face of the end walls measured parallel to the Roadway centerline. When not supporting a Roadway, the Structural Clear Span shall be the widest horizontal opening from interior face to interior face of the end walls measured perpendicular to the Buried Structure centerline.

Structure Class	Structural Clear Span
Class 1	Less than 20.0 feet
Class 2	20.0 feet and greater

6-20.3(1)C General Design Criteria

Buried Structures and associated headwalls, wingwalls and connected barriers, rails and fall protection shall be designed in accordance with the WSDOT *Geotechnical Design Manual* M 46-03, WSDOT *Hydraulics Manual* M 23-03, WSDOT *Bridge Design Manual LRFD* M 23-50, AASHTO LRFD Bridge Design Specifications, AASHTO LRFD Road Tunnel Design and Construction Guide Specifications, Sections 10.8, AASTHO Technical Manual for Design and Construction of Road Tunnels – Civil Elements, Section 13, and the AASHTO LRFD Bridge Construction Specifications. The AASHTO operational classification load modifier for the buried structure shall be that for typical bridges unless noted otherwise.

Buried Structures shall be designed for a minimum Service Life of 75-years. Corrosion and abrasion shall be considered as specified in the WSDOT *Bridge Design Manual LRFD* M 23-50.

Class 2 Buried Structures and associated headwalls and wingwalls, as defined in Section 6-20.1(1), shall include seismic design and mitigation of seismic geological hazards in accordance with the WSDOT *Bridge Design Manual LRFD* M 23-50 and WSDOT *Geotechnical Design Manual* M 46-03. This includes, at a minimum, design for seismic effects in accordance with the AASHTO LRFD Road Tunnel Design and Construction Guide Specifications, Section 10.8, and the AASHTO Technical Manual for Design and Construction of Road Tunnels – Civil Elements, Chapter 13. The AASHTO LRFD Bridge Design Specifications exemption from seismic loading shall not apply.

6-20.3(1)D Geotechnical Considerations

The Contractor shall use the Geotechnical Report prepared for the Buried Structure and available through the source(s) specified in the Special Provisions under Section 1-02.4(2).

The Contractor shall complete all additional geotechnical investigations and designs necessary for this Work as required by the WSDOT *Geotechnical Design Manual*. This includes but is not limited to performing additional permitting, surveying, field investigation, subsurface borings, analysis and modeling. The type, compacted density, and strength properties of the fill adjacent to the Buried Structure shall be established.

If the Geotechnical Report prepared for this Contract does not provide recommendations for the Contractor's selected foundation or wall types, the Contractor shall submit Type 3E Working Drawings consisting of a supplemental Geotechnical Report for all foundation and wall types selected which are not provided for in the recommendations.

6-20.3(1)E Hydraulic Considerations

The Contractor shall complete all additional hydraulic investigation and design necessary for this Work as required by the WSDOT *Hydraulics Manual*. This includes but is not limited to performing additional permitting, surveying, field investigation, subsurface borings, analysis and modeling.

All Buried Structures and associated headwalls and wingwalls shall be designed for scour from the design flood (100 year flood event) and the check flood (500 year flood event) in accordance with the WSDOT *Bridge Design Manual LRFD* and the AASHTO LRFD Bridge Design Specifications unless additional design criteria is documented in the Final Hydraulic Design report. Channel lateral migration shall be considered.

Buried Structures and associated headwalls and wingwalls shall include the effects of scour in accordance with the WSDOT Bridge Design Manual LRFD. Alternative methods of protection against scour may be used with the concurrence of the Engineer.

6-20.3(1)F Worker, Pedestrian, and Bicycle Fall Protection

For Buried Structures and associated headwalls and wingwalls, worker, pedestrian and bicycle fall protection shall be provided in accordance with the WSDOT Design Manual, and shall be designed in accordance with the WSDOT Bridge Design Manual LRFD.

6-20.3(1)G Traffic Barrier and Guardrail

Traffic barrier shall be designed for a minimum Test Level Four (TL-4) impact load, unless otherwise specified.

When traffic barrier or guardrail is connected to the Buried Structure, the supporting Structure shall be designed for the loads transferred from the barrier or guardrail to the Structure.

6-20.3(1)H Concrete Structures

When the Buried Structure is located in a marine or non-marine corrosive environment as defined in the WSDOT *Bridge Design Manual LRFD*, corrosion-resistant reinforcement defined in the WSDOT *Bridge Design Manual LRFD* shall be used. The minimum cover requirements for direct exposure to salt water and coastal situations of the AASHTO LRFD Bridge Design Specifications shall apply.

When the fill depth is less than 2.0 feet at any point above the Buried Structure and is supporting a Roadway, it shall be considered directly exposed to vehicular traffic in accordance with the WSDOT Bridge Design Manual LRFD.

All Work for constructing Class 4000D concrete topping slab shall meet the requirements specified in Section 6-02.3. Bridge approach slabs shall be required for Buried Structures as specified in the WSDOT Bridge Design Manual LRFD.

6-20.3(1)I Structural Plate Structures

Steel structural plate shall not be used in locations conforming to marine or non-marine: corrosive environments as defined in Chapter 6 of the WSDOT *Bridge Design Manual LRFD*. When using galvanized or zinc coated metal structural plate Structures below the Hydraulic Design Flood Elevation, a reinforced concrete splash wall is required. See the WSDOT Bridge Design Manual LRFD for additional details and requirements.

Minimum backfill cover over the top of the Structure and the minimum backfill width on each side of the Structure shall be in accordance with the AASHTO LRFD Bridge Design Specifications.

6-20.3(2) Submittals

6-20.3(2)A Plans, Specifications and Calculations

For a Contractor Supplied Design the Contractor shall submit Type 2E Working Drawings, with a 30-day review period, consisting of site specific plans, specifications, supporting calculations and fabrication shop drawings. All components of the Buried Structure system including but not limited to, the Structure, associated headwalls, wingwalls and other components shall be submitted simultaneously as a comprehensive submittal.

The Contractor shall be responsible for carrying out a thorough check of the Plans and calculations in accordance with WSDOT Bridge Design Manual LRFD Section 1.4.6, and shall include documentation of compliance with Bridge Design Manual LRFD Appendix 1.4-A2 in the buried structure Working Drawing submittal. When a submittal is marked “revise and resubmit”, the Contractor shall respond to all reviewer comments and provide justification for any comments that are not incorporated or addressed in the resubmitted Working Drawing.

The recommendations and requirements contained in the Geotechnical Report and Final Hydraulic Design report for the Structure shall be met.

At a minimum, the site-specific plans and specifications shall include:

1. A plan view, an elevation view, a cross section and complete site-specific details for each structure. Plan details may be used but shall be modified to show all the geometric information necessary to fabricate, construct, and place the Structure including all alignments, horizontal and vertical curve data, offsets, dimensions, elevations, profiles, and grades. Final ground lines shall be shown.
2. Materials, equipment, and installation methods.
3. Cross sections showing the Structure and the proposed material within, and extents of, the Zone of Influence. In fill sections, the cross sections shall show the limits and extent of backfill material placed above original ground.
4. All details for foundations and leveling pads, including details for steps in the foundations or leveling pads.
5. All details for construction of the approach slabs, headwalls, wingwalls, roadway de-icing salt and chloride protection and other associated components.
6. Copies of all applicable Standard Plans or any element(s) thereof intended to be utilized as part of the Contractor Supplied Design.

At a minimum, the calculations shall:

1. State all applicable design criteria, and provide references to sources of design requirements.
2. Clearly describe geotechnical design parameters, groundwater conditions, hydraulic and scour design parameters, sequencing considerations, and governing assumptions.

3. Include load and surcharge conditions associated with geometry, adjacent structures, construction, and roadways as shown in the Plans. Construction loads or surcharges shall be provided by the Contractor or verified by the Contractor.
4. Address all aspects of Service, Strength, and Extreme Event Limit States, including load and resistance factors and stability checks.
5. Include detailed explanations of all symbols, design input, material property values, design tables, and computer program input and output.
6. Demonstrate how the minimum Service Life of 75-years shall be achieved.
7. Exclude computations for elements covered by the Standard Plans.

At a minimum, the fabrication shop drawings:

1. For precast concrete units, shall be in accordance with Section 6-02.3(9)A and shall contain the installation and backfill procedure(s).
2. For structural plate steel and other steel components, shall be in accordance with Section 6-03.3(7).
3. May be submitted as a separate Type 2 Working Drawing following approval of the Type 2E Working Drawing's submittal.

6-20.3(2)B Load Rating Report

For a Contractor Supplied Design, for a Class 2 Buried Structure, the Contractor shall submit a Load Rating Report except when the effects of live load may be neglected in accordance with the WSDOT Bridge Design Manual LRFD.

The Load Rating Report shall be submitted as a Type 2E Working Drawing prepared in accordance with the AASHTO Manual for Bridge Evaluation and the WSDOT *Bridge Design Manual LRFD*, Chapter 13. Soil parameters shall be in accordance with the design requirements, the WSDOT *Geotechnical Design Manual* and the Geotechnical Report prepared for the project.

If the Contractor elects to use Buried Structure Standard Plans, a Load Rating Report is not required to be submitted.

6-20.3(2)C Dewatering System

If water is expected to be present in the excavation, or is found to be present once excavation begins, the Contractor shall submit a Type 2 Working Drawing consisting of a dewatering plan.

6-20.3(2)D Manufacturer's Installation Instructions

For Class 1 structural plate Structures and other Class 1 proprietary Buried Structure systems, the Contractor shall submit a Type 1 Working Drawing consisting of the manufacturer's installation instructions for the Structure prior to the preconstruction conference and construction. The manufacturer's instructions shall provide step-by-step directions for construction of the Structure.

6-20.3(2)E Installation Plan

For Class 2 Buried Structures, prior to the preconstruction conference, the Contractor shall submit a Type 2E Working Drawing consisting of an installation plan, including the manufacturer's installation instructions, Working Drawings and substantiating calculations. The installation plan shall cover all aspects of installation, including but not limited to native subgrade preparation, bedding, leveling, and foundation construction, unit identifier and placement location, assembly, how alignment of the units will be maintained and how adjustments to alignment will be accomplished, how units will be pulled together to attain the proper joint gap, bolting requirements, backfilling requirements and shape control during backfilling. The installation plan shall address all bracing requirements and how the Structure is monitored during and after construction and backfilling to ensure the finished product meets all design and construction requirements and all geometric tolerances. Minimum backfill cover over the structure to support construction equipment loadings shall be specified.

6-20.3(3) Tolerances

Tolerances for cast-in-place concrete components of Buried Structures shall be in accordance with Section 6-02.3(7). Reinforcement placement shall meet the tolerances specified in Section 6-02.3(24)C.

For the Buried Structure location:

1. Horizontal deviation from alignment or work line: ± 2.0 inches.
2. Vertical deviation from profile grade: ± 1.0 inch.

6-20.3(3)A Concrete Structures

Precast concrete Buried Structures shall conform to the fabrication tolerances of Section 6-02.3(9)F.

Fabrication tolerances for precast concrete three-sided Structures shall be as follows:

1. Internal Dimensions: The internal dimension shall not vary more than 1.0 percent or 2.0 inches, whichever is less, from the specified dimensions. The haunch dimensions shall not vary more than ¾ inch from the specified dimensions.
2. Slab and Wall Thickness: The slab and wall thickness shall not be less than that specified by more than 5.0 percent or ½-inch, whichever is greater. A thickness more than that specified will not be a cause for rejection if proper joining is not affected.
3. Length of Opposite Surfaces: Variations in lengths of two opposite surfaces of the three-sided section shall not be more than ¾-inch unless beveled sections are being used to accommodate a curve in the alignment.

Fabrication tolerances for precast concrete box and precast concrete split box Structures shall be as follows:

1. Internal Dimensions: The internal dimensions shall not vary more than 1 percent from the specified dimensions. If haunches are used, the haunch dimensions shall not vary more than ¼ inch from the specified dimensions.
2. Slab and Wall Thickness: The slab and wall thickness shall not be less than that specified by more than 5.0 percent or 3⁄16-inch, whichever is greater. A thickness more than that specified will not be a cause for rejection.
3. Length of Opposite Box Segments: Variations in lengths of two opposite surfaces of the box segments shall not be more than ⅛-inch per foot of internal span, with a maximum of ⅝-inch for all sizes through 7.0 feet internal span, and a maximum of ¾ inch for internal spans greater than 7.0 feet, except where beveled sections are being used to accommodate a curve in the alignment.
4. Length of Box Segments: The underrun in length of a segment shall not be more than ⅛-inch per foot of length with a maximum of ½-inch in any one box segment.
5. Length of Legs and Slabs: The variation in length of the legs shall not be more than ⅛-inch per foot of the rise of the leg per leg with a maximum of ⅝-inches. The differential length between opposing legs of the same segment shall not be more than ½-inch. Length of independent top slab spans shall not vary by more than ⅛-inch per foot of span of the top slab, with a maximum of ⅝-inches.

Placement and erection tolerances for precast components shall be as follows:

1. Maximum offset in alignment of matching edges: ± ½ inches, not to exceed the width of the joint.
2. Joint width: ± ½ inches

6-20.3(3)B Structural Plate Structures

Tolerances for structural plate pipes, arches and boxes shall be in accordance with the AASHTO LRFD Bridge Construction Specifications Section 26 and the manufacturer's recommendations, whichever is more restrictive.

All Class 2 structural plate Buried Structures shall meet the structure dimension tolerances for the assembly of long span structures defined in the AASHTO LRFD Bridge Design Specifications.

6-20.3(4) Preconstruction Conference

Preconstruction conferences are required for all Buried Structures. Class 1 Buried Structures conferences may be omitted when approved by the Engineer.

All submittals required in Section 6-20.3(2), except for the Section 6-20.3(2)B Load Rating Report, shall be accepted and/or approved at least 5 working days before holding the preconstruction conference.

The preconstruction conference shall be held at least 5 working days prior to the Contractor beginning Buried Structure construction at the site to discuss safety, maintenance of traffic, environmental compliance, construction procedures, critical functions during backfilling, quality control steps to control loads, shape and movement, personnel, equipment to be used, and other elements of construction. Those attending shall include:

Representing the Contractor: The superintendent, on site supervisors, and all forepersons in charge of safety, traffic, environmental, excavation, structure construction, and backfilling.

Representing the Manufacturer: For Class 2 Buried Structures, a qualified and experienced manufacturer's representative conforming to Section 6-20.3(4)A.

6-20.3(4)A Manufacturer's Representative

A manufacturer's representative shall be provided for Class 2 Buried Structures. The manufacturer's representative shall be a qualified and experienced representative able to resolve construction problems. The manufacturer's representative shall be present at the preconstruction conference, on site during initial installation and available at other times as needed throughout construction. Recommendations made by the manufacturer's representative shall be followed by the Contractor unless the recommendations deviate from the Contract requirements. In the instance where a recommendation deviates from the Contract, the Contractor shall request acceptance from the Engineer for the Change.

6-20.3(5) Excavation

Excavation shall conform to Section 3-07.3(3). The Contractor shall excavate to the lines and grades identified in the Plans and Working Drawings. The excavation limits shall be increased to account for the placement of Buried Structure bedding and leveling materials and structural backfill.

Material at the bottom of the excavation that is unstable or unsuitable shall be removed in accordance with Section 3-07.3(1)C. If the excavation is dry enough that replacement material can be compacted without causing pumping or further degradation of the material exposed in the bottom of the excavation (after unstable removal), the unstable material shall be replaced in accordance with Section 3-07.3(1)C. Quarry spalls may be used in the overexcavation area if needed but shall be capped with a minimum of 6 inches of Crushed Surfacing Base Course (CSBC).

If the bottom of the excavation is too wet for compaction to occur, the Contractor shall place Construction Geosynthetic for Soil Stabilization - Woven, conforming to Section 9-33.2(1), on the exposed bottom of the excavation. Geosynthetic shall be overlapped as required by the manufacturer, but not less than 1.0 foot at seams. The Contractor shall stretch out the Geosynthetic to ensure that no slack or wrinkles exist in the geosynthetic prior to backfilling. Backfill consisting of CSBC or AASHTO Grading No. 57 shall be placed on top of the Geosynthetic to reestablish lines and grade. Compaction of the CSBC or AASHTO Grading No. 57 shall be by static methods or by track walking and shall impart only enough energy to seat the granular materials together and provide a stable, non-shifting, working surface. Controlled Density Fill (CDF) or lean concrete may be used with no geosynthetic and no compaction.

Upon completing the excavation, the Contractor shall notify the Engineer. No other permanent part of the Buried Structure or associated headwalls or wingwalls shall be placed until the Engineer has given permission to proceed.

If water is present within the excavation, the Contractor shall dewater the excavated area in accordance with the Section 6-20.3(2)C Working Drawing submittal before placing the bedding material.

Existing structures and obstructions shall be removed in accordance with Section 3-02.3.

6-20.3(5)A Construction Dewatering

The Contractor shall design, install, operate, maintain, and remove a construction dewatering system. The construction dewatering system shall be used to remove precipitation from the work area, surface water that enters the work area, and seepage when excavations extend below groundwater. The system shall be capable of handling surface water, precipitation, and groundwater flow associated with seasonal groundwater variations and storm events. The system shall provide for a reasonably dry excavation free of standing water that impedes construction or degrades the working surface of the excavation. Discharge from the dewatering system shall be handled in accordance with Section 8-01.3(1)C1.

6-20.3(6) Bedding and Foundations

6-20.3(6)A Bedding and Leveling

Buried Structure bedding material shall be placed in accordance with the Contract documents and the submittals of Sections 6-20.3(2)A, 6-20.3(2)D, and 6-20.3(2)E.

Cast-in-place and precast reinforced concrete foundation elements require a 6.0-inch minimum thickness layer of Buried Structure bedding material, defined as either CSBC or AASHTO Grading No. 57. In addition to the bedding material, precast reinforced split box Structures require a 1-inch minimum to 2-inch maximum leveling course on top of the bedding. The leveling course shall meet the requirements of Section 9-03.1(2)B for Grading Class 1 or Class 2 sand. Alternative leveling material may be submitted for acceptance by the Engineer, but in no cases shall the particle size be greater than ⅜-inch. The plan limits of the Buried Structure bedding and leveling material, when applicable, shall extend 1.0 foot beyond the plan limits of the foundation or the Structure as applicable. The Buried Structure bedding and leveling material shall be compacted in accordance with the Section 3-07.3(1)E requirements for backfill supporting Structures. If the bedding material has voids after compaction, then a Filter Fabric, conforming to Section 9-08.6, shall be placed over the bedding to separate the leveling course from the bedding material. Leveling material shall not be placed under precast reinforced concrete retaining walls.

All Buried Structure bedding material adjacent to structural plate structures shall meet the material requirements for backfill material in Section 6-20.3(9).

Structures with a curved, nonplanar bottom, such as structural plate pipes, require Buried Structure bedding material with a minimum thickness of twice the corrugation depth, but not less than 6.0 inches. The Buried Structure bedding material shall be shaped to conform to the bottom of the structure and shall provide a uniform bearing throughout the Buried Structure length. The Buried Structure bedding material shall be centered beneath the Structure and shall have a minimum width of one-third the Structural Clear Span for horizontal elliptic shapes and one-half the Structural Clear Span otherwise. Bedding material shall be compacted in accordance with Section 3-07.3(1)E except that the top layer of bedding (thickness of outside diameter of the pipe minus the inside diameter of the pipe) shall be loosely placed (not compacted), to cushion the invert and allow corrugations to nest or seat into it.

Rock, in either ledge or boulder formation, hard pan, or cemented gravel occurring in the base material shall be excavated to below the Buried Structure bedding material.

6-20.3(6)B Foundations

Cast-in-place concrete foundations and components of foundations shall be constructed in accordance with Section 6-02.

Precast concrete foundations shall be constructed in accordance with Section 6-02.3(9).

6-20.3(7) Fabrication

Welding of steel shall be in accordance with Section 6-03.3(25). Welding of steel structural plate Structures shall comply with the AWS D1.1/D1.1M Structural Welding Code.

Welding of aluminum shall conform to the ANSI/AWS D1.2/D1.2M Structural Welding Code – Aluminum.

6-20.3(7)A Precast Concrete Structures

Except as otherwise noted by these specifications, precast concrete buried structures shall conform to all requirements of Section 6-02.3(9).

Precast prestressed units shall be fabricated and transported in accordance with Section 6-02.3(25).

For Class 1 and Class 2 precast concrete, three-sided and split box Structures, unless otherwise shown in the Plans, the Contractor shall, at a minimum for each set of forms used, progressively shop assemble the top and bottom units of the first three adjacent segments for inspection of fit up. The installing Contractor shall observe the shop fit up. The date and time of the shop fit up shall be scheduled during normal business hours and communicated to both the installing Contractor and the Engineer. As an alternative to physically being present to observe the fit-up, the Contractor and the Engineer may agree to observe the fit-up via video conference. Units shall not be disassembled prior to receiving the Engineer's acceptance. If the Engineer accepts the initial assembly then no additional shop assembly will be required unless the Contractor changes forms, the forms show signs of damage, or there is a geometric change to the forms. If issues are found during the progressive shop assembly, the Contractor shall make corrections and continue the progressive shop assembly until three, consecutive segments have been successfully shop assembled. The shop assembly shall be done on a flat level surface at the fabrication plant. Bunking or shimming shall not be allowed during the shop assembly.

The following information shall be legibly and permanently marked on one inside face of each precast unit by indentation, waterproof paint or other means acceptable to the Engineer:

1. Span and rise dimensions
2. Date of fabrication
3. Name or trademark of the fabricator
4. WSDOT Contract Number
5. Unit identifier shown in the Plans or Working Drawings. If the precast Structure fabricator modifies the finished precast units for shop fit up then the fabricator shall sequentially number all of the precast units for field assembly. The Contractor shall assemble the precast culvert units according to the fabricator's sequential numbering.

6-20.3(7)B Structural Plate Structures

The following information shall be legibly and permanently marked on the inside face of each structural plate by waterproof paint or other means acceptable to the Engineer:

1. Plate thickness or gage
2. Date of fabrication
3. Name or trademark of the fabricator
4. WSDOT Contract Number
5. Unit identifier shown in the Plans or Working Drawings

6-20.3(8) Placement and Assembly

Buried Structures shall be placed and assembled in accordance with the Contract documents and the submittals of Sections 6-20.3(2)A, 6-20.3(2)D, and 6-20.3(2)E.

Components with identified pick points or lifting locations shall be handled using those locations. Component pieces shall be set into their final position in a manner that preserves the lines and grade established for the structure. Base units, precast foundation units, and bearing surfaces may be slid on grade to adjust their position, but if sliding results in grade changes, misalignment, or native subgrade, bedding, or leveling material infiltration within the annular joint space (inhibiting jointing between components), the component shall be removed, the grade adjusted, and foreign material(s) cleared. Grade shall be adjusted such that components can be assembled in accordance with the required tolerances.

Shims used by the Contractor to position components for assembly shall be removed unless the shims are specified as permanent in the Plans or Working Drawings.

Components that are not self-supporting shall be braced or supported by the Contractor during assembly.

Construction of cast-in-place concrete components shall be in accordance with Section 6-02.

6-20.3(8)A Precast Concrete Structures

For Structures that are expected to support traffic within 7 days after the placement of prefabricated segments is complete, the Contractor shall designate an individual to verify and record conformance to the erection tolerances for each segment as it is placed.

Components with weld-ties shall be connected by welding the weld-tie anchors in accordance with Section 6-03.3(25). The welding ground shall be attached directly to the plates being welded. After connecting the weld-tie anchors, the Contractor shall paint the exposed metal surfaces with one coat of zinc-rich primer conforming to Section 9-08.1(2)F or Section 9-08.1(2)N. Keyways shall be filled with Grout Type 2 for Nonshrink Applications, unless specified otherwise.

The Contractor shall install a continuous strip of Butyl Rubber Sealant within all tongue and groove (ship lap) and butt joints prior to connecting the precast elements together. The Butyl Rubber Sealant shall be sized in accordance with the manufacturer's recommendations but shall not have a cross section smaller than ½-inch by 1½-inch.

The Contractor shall wrap all exterior joints along the top and sides of precast concrete structures (except top surfaces that will have a waterproof membrane with an HMA overlay, a concrete overlay or a concrete topping slab) with a 12.0-inch wide strip of External Sealing Band centered about the joint and adhesively bonded to the concrete surface.

6-20.3(8)B Structural Plate Structures

Construction of structural plate pipes, arches and boxes shall conform to the AASHTO LRFD Bridge Construction Specifications, Section 26.

Plates at longitudinal and circumferential seams shall be configured with the seams staggered so that not more than three-barrel plates come together at any one point.

When required, temporary bracing shall be installed and shall remain in place as long as necessary to protect workers and to maintain structure shape during placement and assembly.

Bolts and bolted connections shall conform to the requirements of AASHTO M 167 for steel and AASHTO M 219 for aluminum. Bolts shall be sufficiently torqued to avoid backing out while compacting backfill.

Where aluminum will contact concrete or grout, two coats of paint shall be applied to the aluminum at the contact surface in accordance with Section 7-08.3(2)D.

Where the galvanized coating on structural plate has been damaged in handling or installing, such damaged areas shall be thoroughly painted with galvanizing repair paint, high zinc dust content.

6-20.3(9) Backfilling

The backfill outside of Buried Structures shall be granular material meeting the requirements in the Working Drawings, meeting the Buried Structure manufacturer's requirements and conforming to the requirements of AASHTO M 145 A-1 or A-3. Granular material shall consist of a crushed rock and/or processed angular material. On site granular soils may be considered for backfill around the Structure if the material meets the requirements in this specification.

The following backfill materials generally meet the AASHTO M 145 A-1 or A-3 requirements. Additional gradation requirements by the Structure manufacturer may apply.

1. Section 9-03.9(1) Ballast
2. Section 9-03.9(3) Crushed Surfacing (Top and Base Course)
3. Section 9-03.9(4) Maintenance Rock
4. Section 9-03.12(1)A Gravel Backfill for Foundations Class A
5. Section 9-03.12(3) Gravel Backfill for Pipe Zone Bedding
6. Section 9-03.14(1) Gravel Borrow
7. Section 9-03.14(2) Select Borrow
8. Section 9-03.14(4) Gravel Borrow for Structural Earth Wall

Backfilling shall conform to Section 3-07.3(1)E and the Working Drawings. Backfill of Buried Structures to the minimum cover level specified for the Structure shall be considered to be supporting the structure for determining backfill placement layer thicknesses and compaction densities. Backfill shall be brought up incrementally on each side of the structure to balance the loading until the top of the Structure is reached. The difference in backfill height on opposing sides of the Structure shall not exceed 2.0 feet unless otherwise allowed by the Engineer.

Equipment used to compact backfill within 3.0 feet from sides of Buried Structures shall have received the Engineer's acceptance prior to use. Sheepsfoot rollers or rollers with protrusions for compacting shall not be used until there is more than 2.0 feet of compacted backfill over the Structure.

Where backfill material is placed against dissimilar materials not meeting backfill material requirements, a suitable geotextile shall be provided to avoid migration.

When specified in the Contract documents or Working Drawings, the Contractor shall place and compact materials within the Buried Structure. The Contractor may place and compact materials prior to finishing assembly provided the placement and compaction does not damage or distort the Structure or hinder the achievement of the specified tolerances.

6-20.3(9)A Backfilling of Structural Plate Structures

The Zone of Influence shall include the structural backfill outside of the Buried Structure within the following limits:

1. Vertically, from the lowest elevation of the structural plate up to the lesser of the minimum cover height over the structural plate, the bottom of asphalt pavement or the top of a reinforced concrete pavement as defined in the AASHTO LRFD Bridge Design Specifications.
2. Horizontally, between the outer edges of the minimum structural backfill widths on each side of the structural plate as defined in the AASHTO LRFD Bridge Design Specifications.

All water and backfill materials inside or adjacent to a structural plate Buried Structure or within the Zone of Influence shall meet the following requirements:

1. pH in accordance with WSDOT T 417 shall be within the range of 6.0 and 10.0 for galvanized steel Buried Structures and within the range of 4.0 and 9.0 for aluminum Buried Structures and shall also meet the requirement to provide a minimum 75 year service life
2. Resistivity in accordance with WSDOT T 417 shall be 2,500 ohm-cm minimum for galvanized steel Buried Structures and 500 ohm-cm minimum for aluminum Buried Structures but shall not be less than that required to provide a minimum 75-year Service Life
3. Chlorides in accordance with AASHTO T 291 shall be 100 ppm maximum
4. Sulfates in accordance with AASHTO T 290 shall be 200 ppm maximum
5. Organics shall be 1.0% or less by dry weight.

When required, temporary bracing shall be installed and shall remain in place as long as necessary to protect workers and to maintain structure shape during backfilling.

Structure shape shall be checked regularly by the Contractor during backfilling to verify acceptability of the construction methods used. The magnitude of allowable shape change shall meet the manufacturer's requirements as well as the requirements in Section 6-20.3(3). For Class 2 Buried Structures, a manufacturer's representative shall assist as described in Section 6-20.3(4)A.

Installation deflection inspections by direct measurement shall be performed by the Contractor immediately after construction and 30 days or more after construction. Inspection results shall be reported to the Contracting Agency within 2 working days of performing the inspection. Installation deflections shall meet the requirements of Section 6-20.3(3).

6-20.3(10) Wingwalls and Headwalls

The Contractor shall construct wingwalls and headwalls associated with Buried Structures in conformance with the Contract documents and Working Drawing submittals.

For Buried Structures crossing water, portions of headwalls and wingwalls below the water surface of the Hydraulic Design Flood Elevation shall be reinforced concrete or have a reinforced concrete fascia.

Structural Earth Wall wingwalls shall not use metallic ground reinforcement below the water surface of the Hydraulic Design Flood Elevation unless the pH, in accordance with WSDOT T 417, of the water in front of the wall and of the groundwater are within the range of 5.0 and 10.0.

Cast-in-place concrete components of headwalls and wingwalls shall be constructed in accordance with Section 6-02.

Precast concrete construction shall conform to Sections 6-02.3(9) and 6-11.3(3).

Bedding material shall be furnished, placed, and compacted in accordance with Section 6-20.3(6).

6-20.4 Vacant

6-20.5 Payment

Payment will be made for each of the following Bid items that are included in the Proposal:

"Agency Designed Buried Structure No. _____", lump sum.

The lump sum Contract price for "Agency Designed Buried Structure No. _____" shall be full payment to perform the Work as specified in Section 6-20.3.

The approximate quantities of materials and work for the lump sum item "Agency Designed Buried Structure No. _____" may be provided in the Contract documents. If so, the quantities listed are only for the convenience of the Contractor in determining the volume of work involved and are not guaranteed to be accurate. Quantities may vary depending on the Contractor's Work methods, order of work, suitability of excavated materials, and structure dimensions. The prospective bidders shall verify these quantities before submitting a bid. No adjustments other than for accepted changes will be made in the lump sum Contract price for "Agency Designed Buried Structure No. _____" even though the actual quantities required may deviate from those listed.

"Contractor Designed Buried Structure No. _____", lump sum.

The lump sum Contract price for "Contractor Designed Buried Structure No. _____" shall be full payment to perform the Work as specified in Section 6-20.3.

The approximate quantities of materials and work for the lump sum item "Contractor Designed Buried Structure No. _____" may be provided in the Contract documents. If so, the quantities listed are only for the convenience of the Contractor in determining the volume of work involved and are not guaranteed to be accurate. Quantities may vary depending on the Contractor's Work methods, order of work, suitability of excavated materials, and structure dimensions. The prospective bidders shall verify these quantities before submitting a bid. No adjustments other than for accepted changes will be made in the lump sum Contract price for "Contractor Designed Buried Structure No. _____" even though the actual quantities required may deviate from those listed.

Payment for Class 4000D concrete topping slab shall be included in the lump sum item, "Contractor Designed Buried Structure No. ____".

"Shoring or Extra Excavation Cl. A _____", lump sum.

6-21 Modified Concrete Overlay - Microsilica or Fly Ash

6-21.1 Description

This Section 6-21 is for Contracts with the bid item “Modified Concrete Overlay” when the Contractor elects to use microsilica modified concrete or fly ash modified concrete for the overlay. If the Contractor elects to use latex modified concrete for the overlay, use Section 6-22, Modified Concrete Overlay - Latex.

This Work consists of surveying, Scarifying the existing concrete bridge deck, removing and replacing unsound concrete below the Scarification Depth, and placing, finishing, curing, and texturing a modified concrete overlay using microsilica modified concrete or fly ash modified concrete.

6-21.1(1) Definitions

Existing Bridge Deck Surface – The surface of the existing bridge deck upon which traffic rides prior to any Work by the Contractor. It includes wheel ruts and other anomalies, if any.

Existing Bridge Deck Surface Model – The three-dimensional model of the Existing Bridge Deck Surface, described by station, offset, and elevation, as required in Section 6-21.3(4), Surveying for Scarification and Modified Concrete Overlay. The Existing Bridge Deck Surface Model is created by the Contractor.

Idealized Existing Bridge Deck Surface Model – This is the surface from which the Contractor shall measure the Scarification Depth. The Engineer will create the Idealized Existing Bridge Deck Surface Model by making Engineer-desired adjustments to the profile and transverse slopes of the Contractor’s Existing Bridge Deck Surface Model. These Engineer desired adjustments may include eliminating wheel ruts, eliminating bird baths, or improving the ride across and off the bridge. The Idealized Existing Bridge Deck Surface Model is composed of centerline station, elevation, and superelevation, and for crown and grade breaks will additionally include offset, elevation, and superelevation. These will be provided for the same centerline and stations supplied by the Contractor in the Existing Bridge Deck Surface Model.

Top of Overlay Model – The three-dimensional model of the required top of finished modified concrete overlay surface, created by the Engineer and provided for the Contractor’s use in setting the finish grades of the concrete finishing machine. The Top of Overlay Model will usually be parallel to the Idealized Existing Bridge Deck Surface Model in order to provide an overlay of uniform thickness. It is composed of centerline station, elevation, and superelevation, and for crown and grade breaks will additionally include offset, elevation, and superelevation. These will be provided for the same centerline and stations provided by the Contractor in the Existing Bridge Deck Surface Model.

Process Wastewater means all non-stormwater which, during manufacturing or processing, comes into direct contact with or results from the production or use of raw material, intermediate product, finished product, byproduct, or waste product. If stormwater commingles with Process Wastewater, the commingled water is considered Process Wastewater. Process Wastewater includes water used by the Contractor, which includes but is not limited to:

1. flushing, misting, water-blasting, saturating, and cleaning,
2. rotary milling, hydro-demolition, and Scarification by any other means,
3. deck repair excavation,
4. sandblasting and dust prevention,
5. curing.

Scarify, Scarification – To break up the surface of the existing bridge deck to specified depths using Contractor selected equipment in accordance with Section 6-21.3(6).

Scarification Depth – The depth identified as such in the plans. It is measured from (a) the Idealized Existing Bridge Deck Surface to (b) the mean depth between peaks and valleys of the deck surface after hydro demolition.

6-21.2 Materials

Materials shall meet the requirements of the following sections:

High Molecular Weight Methacrylate (HMWM) Resin for Crack and Cold Joint Sealing	6-21.2(3)
Sand for Abrasive Finish of Cracks and Cold Joints	6-21.2(3)
Portland Cement	9-01.2(1)
Blended Hydraulic Cement	9-01.2(1)B
Fine Aggregate	9-03.1
Coarse Aggregate	9-03.1
Mortar	9-20.4
Burlap Cloth	9-23.5
Admixtures	9-23.6
Fly Ash	9-23.9
Microsilica Fume	9-23.11
Water for Concrete	9-25.1

6-21.2(1) Modified Concrete

The decision of whether to use microsilica modified concrete or fly ash modified concrete for the modified concrete overlay belongs to the Contractor. However, the Contractor shall not use both on any one bridge.

6-21.2(1)A Microsilica Modified Concrete

Microsilica modified concrete shall be a workable mix, uniform in composition and consistency. Mix proportions per cubic yard of microsilica modified concrete shall be as follows:

Portland Cement Type 1 or Type 2, or Blended Hydraulic Cement Type IL(X)	658 pounds
Microsilica Fume	52 pounds
Fine Aggregate	1,515 pounds
Coarse Aggregate	1,515 pounds
Water/Cementitious Ratio	0.33 maximum
Air (± 1½ percent)	6 percent
Slump	7 inches maximum

Fly ash, ground granulated blast furnace slag, and Type 3 portland cement shall not be used in microsilica modified concrete.

Fine aggregate shall be Class 1. Coarse aggregate shall be AASHTO grading No. 7 or No. 8.

Microsilica admixture shall be either a dry powder or a slurry admixture. Microsilica will be accepted based on submittal of a Manufacturer's Certificate of Compliance in accordance with Section 1-06.3. If the microsilica is a slurry admixture, the microsilica content of the slurry shall be certified as a percent by mass.

The Contractor shall adjust the slump to accommodate the gradient of the bridge deck, subject to the maximum slump specified.

The maximum allowable and actual water/cementitious ratio shall be calculated using all of the available mix water, including water added at the plant, water added in transit and at the job site, water in all admixtures, water in the microsilica slurry if a slurry is used, and the free water in the aggregates but not the water absorbed by the aggregates. The following are considered cementitious materials: portland cement, blended hydraulic cement, and microsilica fume.

All water-reducing and air entraining admixtures, and superplasticizers, shall be used in accordance with the admixture manufacturer's recommendations.

Mixing of microsilica modified concrete shall be in accordance with Section 6-02, with the following exceptions:

1. Microsilica Modified Concrete shall be mixed at a batch plant.
2. The volume of Microsilica Modified Concrete transported by truck shall not exceed 6-cubic yards per truck.

6-21.2(1)B Fly Ash Modified Concrete

Fly ash modified concrete shall be a workable mix, uniform in composition and consistency. Mix proportions per cubic yard of fly ash modified concrete shall be as follows:

Portland Cement Type 1 or Type 2, or Blended Hydraulic Cement Type IL(X)	611 pounds
Fly Ash	275 pounds
Fine Aggregate	38 percent of total aggregate
Coarse Aggregate	62 percent of total aggregate
Water/Cementitious Ratio	0.30 maximum
Air (± 1½ percent)	6 percent
Slump	7 inches maximum

Fly ash shall be Class F only.

The Contractor may substitute 886 pounds of blended hydraulic cement Type IP(X)MS for the following: 611 pounds of portland cement Type 1 or 2 and 275 pounds of fly ash. Only Type IP(X)MS that is blended with Class F fly ash is permitted for use. Type IP(X)MS that is blended with natural pozzolans, granulated blast furnace slag, or Type 3 portland cement are not allowed.

Ground granulated blast furnace slag and Type 3 portland cement shall not be used in fly ash modified concrete. Fly ash shall not be used in blended hydraulic cement Type IL(X).

Fine aggregate shall be Class 1. Coarse aggregate shall be AASHTO grading No. 7 or No. 8.

The Contractor shall adjust the slump to accommodate the gradient of the bridge deck, subject to the maximum slump specified.

The maximum allowable and actual water/cementitious ratios shall be calculated using all the available mix water, including water added at the plant, water added in transit and at the job site, water in all admixtures, and the free water in the aggregates but not the water absorbed by the aggregates. The following are considered cementitious materials: portland cement, blended hydraulic cement, and fly ash.

All water-reducing and air entraining admixtures, and superplasticizers, shall be used in accordance with the admixture manufacturer's recommendations.

Mixing fly ash modified concrete shall be in accordance with Section 6-02, with the following exceptions:

1. Fly ash modified Concrete shall be mixed at a batch plant.
2. The volume of fly ash modified Concrete delivered shall not exceed 6-cubic yards per truck.

6-21.2(2) Concrete Class M

Concrete Class M shall be proportioned in accordance with the following mix design:

Portland Cement Type 1 or Type 2, or Blended Hydraulic Cement Type IL(X)	705 pounds
Fine Aggregate	1,280 pounds
Coarse Aggregate	1,650 pounds
Water/Cement Ratio	0.37 maximum
Air (± 1½ percent)	6 percent
Slump (± 1 inch)	5 inches

The following shall not be used in concrete Class M or in blended hydraulic cement Type IL(X): fly ash, ground granulated blast furnace slag, and Type 3 portland cement.

Fine aggregate shall be Class 1. Coarse aggregate shall be AASHTO grading No. 7 or No. 8.

The use of a water-reducing admixture conforming to AASHTO M 194 Type A will be required to produce Concrete Class M with the desired slump, Air entraining admixtures shall conform to AASHTO M 154. The use of accelerating admixtures or other types of admixtures is not allowed.

Concrete Class M shall be mixed in batch-plants and delivered in ready-mix trucks conforming to Section 6-02.3(4)A.

The maximum allowable and actual water/cementitious ratios shall be calculated using all the available mix water, including water added at the plant, water added in transit and at the job site, water in all admixtures, and the free water in the aggregates but not the water absorbed by the aggregates. The following are considered cementitious materials: portland cement and blended hydraulic cement.

6-21.2(3) Material for Sealing Cracks and Cold Joints

High Molecular Weight Methacrylate (HMWM) resin for crack and cold joint sealing shall conform to the following:

Viscosity	<25 cps (Brookfield RVT with UL adaptor, 50 rpm at 77°F) … California Test 434
Density	8.5 to 8.8 pounds per gallon at 77°F… ASTM D2849
Flash Point	>200°F, PMCC (Pinsky-Martens CC)
Vapor Pressure	<0.04 inches Hg at 77°F, ASTM D323
Tg (DSC)	>136°F, ASTM D3418
Gel Time	60 minutes minimum

The promoter/initiator system for the methacrylate resin shall consist of a metal drier and peroxide.

Sand for abrasive finish of the sealed cracks and cold joints shall be crushed sand, oven dried, and stored in moisture proof bags. The sand shall conform to the following gradation:

Sieve Size	Percent Passing	
	Minimum	Maximum
No. 10	98	100
No. 16	55	75
No. 20	30	50
No. 30	8	25
No. 50	0	5
No. 100	0	3

All percentages are by weight.

The HMWM resin shall be stored in a cool dry place and protected from freezing and exposure to temperature in excess of 100°F. The promoter and initiator, if supplied separate from the resin, shall not contact each other directly. Containers of promoters and initiators shall not be stored together in a manner that will allow leakage or spillage from one to contact the containers or material of the other.

6-21.3 Construction Requirements

6-21.3(1) Sequence of Work

The general sequence of the Work shall be as follows:

1. Survey for Modified Concrete Overlay
2. Calculate the Idealized Existing Bridge Deck Surface Model and Top of Overlay Model (by the Engineer)
3. Scarify Existing Bridge Deck Surface
4. Perform Type 1 and Type 2 Deck Repair
5. Clean and blast the finished surface
6. Place and cure the modified concrete overlay
7. Check for bond and repair overlay
8. Crack sealing and texturing

6-21.3(2) Equipment

6-21.3(2)A Rotary Milling Machines

Rotary milling machines shall conform to the following:

1. Shall have a maximum operating weight of 50,000 pounds.
2. Shall be operated with automatic controls and sensors, sensing the grade from an outside reference line, or referencing device, and maintaining the desired transverse slope (expressed as a percentage) within a tolerance of plus or minus 0.1 percent.
3. Shall be capable of producing a finished surface with a tolerance of +0.01, -0.02 feet.
4. Shall be capable of achieving the planed surface profile or superelevation in one pass.
5. The cutting tooth spacing on the rotary milling head shall be less than or equal to ¼ inch.
6. The rotary milling machine shall have cutting teeth that leave a uniform plane surface at all times.
7. All cutting teeth shall be kept at a maximum differential tolerance of ⅜-inch between the shortest and longest tooth, as measured by a straight edge placed the full width of the rotary milling head.
8. Cutting teeth shall be replaced when 30 percent of the total length of the cutting tooth material remains. Prior to each day's use, the Contractor shall confirm to the satisfaction of the Engineer that the cutting teeth are within the specified tolerances.

6-21.3(2)B Hydro-Demolition Machines

6-21.3(2)B1 Hydro-Demolition Machine Requirements

Hydro-demolition machines shall conform to the following:

1. Hydro-demolition machines shall consist of filtering and pumping units operating in conjunction with a remote-controlled robotic device, using high-velocity water jets to remove sound concrete.
2. The hydro-demolition machine shall remove sound concrete within a tolerance of ±¼ inch of the Scarification Depth. Concrete removal exceeding this tolerance below the Scarification Depth will be acceptable provided the concrete removed below the Scarification Depth is unsound concrete.
3. When the specifications require hydro-demolition with rotary heads, the Contractor shall convert all cutting heads to rotary heads if not already so equipped.
4. Hydro demolition machines shall clean all exposed reinforcing steel of all rust and corrosion products.

6-21.3(2)B2 Hydro-Demolition Trial Area Test

Prior to being approved for production use, the Contractor shall prepare a trial area in the bridge deck to be overlaid to demonstrate that the hydro-demolition equipment and methods of operation are capable of producing results satisfactory to the Engineer. The trial area shall consist of a patch of approximately 30 square feet in sound concrete as determined by the Engineer. Satisfactory results include, but are not limited to, concrete removal within a tolerance of ±¼ inch of the Scarification Depth in sound concrete.

The equipment shall be programmed to remove concrete to the Scarification Depth shown in the Plans, but no more than 2 inches with a single pass of the machine. The Engineer will grant acceptance of the equipment based on successful results from the trial area test.

6-21.3(2)C Power Driven Hand Tools

Power driven hand tools are limited to the following:

1. Jack hammers no heavier than the nominal 30-pound class.
2. Chipping hammers no heavier than the nominal 15-pound class.
3. Other mechanical means acceptable to the Engineer.

Power driven hand tools shall not be operated at angles greater than 45 degrees as measured from the surface of the deck to the tool.

6-21.3(2)D Air Compressor

Air compressors shall be equipped with oil traps to eliminate oil from being blown onto the bridge deck.

6-21.3(2)E Vacuum Machine

Vacuum machines shall be capable of collecting all dust, concrete chips, freestanding water and other debris encountered while cleaning after Scarifying and after Type 1 and Type 2 Deck Repair. The machines shall be equipped with collection systems that allow the machines to be operated in air pollution sensitive areas and shall be equipped to not contaminate the deck during final preparation for concrete placement.

6-21.3(2)F Water Delivery System

The Contractor shall apply water at the pressures and volumes required for flushing, misting, saturating, and water-blasting. All water applied to the deck for these purposes shall be potable water and shall not be re-used. The water delivery system shall include a portable high-pressure sprayer. The sprayer shall be readily available to all parts of the deck being overlaid and shall be able to discharge water in a fine mist to prevent accumulation of free water on the deck, and for water-blasting concrete to remove laitance, rust, stains, dust, and loose paste. Sufficient water shall be available to thoroughly soak the deck being overlaid and to keep the deck wet prior to concrete placement.

The Contractor shall certify that the units used meet the requirements in the table below:

Minimum Requirements for Water Delivery	
Activity	**Water Pressure**
Flushing	none specified
Misting	2,500 psi
Water Blasting	7,500 psi
Saturating	none specified

6-21.3(2)G Finishing Machine

The finishing machine shall meet the following requirements:

1. Shall be self-propelled and capable of forward and reverse movement under positive control.
2. Shall be equipped with augers, followed by an oscillating, vibrating screed, vibrating roller tamper, or a vibrating pan, followed by a rotating cylindrical double drum screed.
3. Shall have the necessary adjustments to produce the required cross section, line, and grade, including the ability to recreate transverse grade breaks within 6 inches left or right of existing transverse grade breaks.
4. Shall be capable of raising the screeds, augers, and all other parts of the finishing mechanical operation to clear the screeded surface and return to the specified grade under positive control.
5. The vibrating screed, roller tamper or pan shall be of sufficient length and width to properly consolidate the mixture.
6. The vibrating frequency of the vibrating screed, roller tamper or pan shall be variable with positive control.
7. For locations on the Bridge Deck 10 foot in width or less, where the conditions do not allow the use of the conventional configuration finishing machines, the Contractor shall use a hand-operated motorized power screed such as a "Texas" or roller screed. The screed shall be capable of finishing the overlay to the same standards as the finishing machine.

6-21.3(3) Submittals

The Contractor shall submit the following Working Drawings in accordance with Section 1-05.3:

1. A Type 1 Working Drawing consisting of catalog cuts and operating parameters of the hydro-demolition machine selected by the Contractor, including the locations and proposed Scarification Depths.
2. A Type 1 Working Drawing consisting of catalog cuts, operating parameters, axle loads, and axle spacing of the rotary milling machine, including the locations and proposed Scarification Depths, if proposed for use on this project.

3. A Type 2 Working Drawing of the Process Wastewater Containment and Disposal Plan. The Process Wastewater Containment and Disposal Plan shall describe all provisions for the containment, collection, and disposal of all Process Wastewater and associated contaminants (including runoff water and contaminants escaping through breaks in the bridge deck) generated by all operations.
4. A Type 2 Working Drawing of the Debris Containment and Disposal Plan. This plan shall describe the methods and materials used to contain, collect, and dispose of all concrete debris generated by all operations, including but not limited to Scarifying, Type 1 Deck Repair, Type 2 Deck Repair, placing and misting the modified concrete overlay, curing the concrete overlay, sandblasting, water-blasting, and cleaning. The Working Drawing shall also address provisions for protecting adjacent traffic from flying debris.
5. A Type 2 Working Drawing of the mix design meeting the requirements of microsilica modified concrete or fly ash modified concrete, and Class M concrete. The Contractor's submittal of a mix design shall be on WSDOT Form 350-040 and shall provide a unique identification for each mix design. A unique identification for the mix design is composed of the combination of the Mix Design Number and the Concrete Plant Number.
6. A Type 2 Working Drawing of the paving equipment specifications and details of the screed rail support system, including details of anchoring the rails and providing rail continuity.
7. A Type 2E Working Drawing of the survey data collected as required in Section 6-21.3(4) and a Type 1 Working Drawing of each day's survey record.
8. A Type 1 Working Drawing of the measurements documenting the deck repair areas as required by Section 6-21.3(7)B.

6-21.3(4) Surveying for Scarification and Modified Concrete Overlay

Prior to beginning Scarifying or otherwise removing materials from the Existing Bridge Deck Surface, the Contractor shall complete a survey of the Existing Bridge Deck Surface to establish the Existing Bridge Deck Surface Model (station, offset, and elevation) to form the basis for the Engineer to create the Idealized Existing Bridge Deck Surface Model (station, offset, elevation) and, from that, the Top of Overlay Model (station, offset, and elevation).

The Engineer will provide the Contractor with primary survey control information consisting of descriptions of two primary control points used for the horizontal and vertical control. Primary control points will be described by reference to the bridge or project-specific stationing and elevation datum. The Engineer will also provide horizontal coordinates for the beginning and ending points and for each Point of Intersection (PI) on each centerline alignment included in the project. The Contractor shall provide the Engineer 21 calendar days' notice in advance of the Contractor's scheduled surveying work to allow the Engineer time to provide the primary survey control information.

The Contractor shall verify the primary survey control information furnished by the Engineer and shall expand the survey control information to include secondary horizontal and vertical control points as needed for the project. The Contractor's survey records shall include descriptions of all survey control points, including coordinates and elevations of all secondary control points.

The Contractor shall maintain detailed survey records, including a description of the work performed on each shift, the methods utilized to conduct the survey, and the control points used. The record shall be of sufficient detail to allow the survey to be reproduced. A Type 1 Working Drawing of each day's survey record shall be provided to the Engineer within 3 working days after the end of the shift. The Contractor shall compile the survey information in an electronic file format acceptable to the Engineer (Excel spreadsheet format is preferred).

Survey information collected by the Contractor to create the Existing Bridge Deck Surface Model shall include the station, offset, and elevation of every point of intersection on a grid created as follows:

1. Longitudinally, for the full length of the bridge and extending 100 feet beyond the bridge back of pavement seat on each end. Data shall be gathered every 20 feet, at even 20-foot station intervals, and also at the centerline of each bridge expansion joint.
2. Transversely, for the full width of the bridge to be overlaid, from curb line to curb line. Data shall be gathered at every longitudinal station centerline, offset at each lane, offset at all changes in superelevation such as crowns and broken-backs, and offset at each curb line.

When traffic wear, such as wheel rutting, is not centered on and parallel to existing lane lines (e.g., when the lane striping has been shifted or skewed at some point during the bridge history), the Contractor shall orient its surveying centerlines, or select offsets at the former lane lines, so that the survey data consistently picks up the high spots at the former lane lines, and not any low spots in the wheel ruts.

The Contractor shall ensure a surveying accuracy to within ± 0.01 feet for vertical control and ± 0.2 feet for longitudinal and transverse control.

The Engineer will establish the Idealized Existing Bridge Deck Surface Model and the Top of Overlay Model based on the Contractor's Existing Bridge Deck Surface Model, and will provide both to the Contractor within five Working Days after receiving the Contractor's survey information.

Except for (1) the primary survey control information, (2) the Idealized Existing Bridge Deck Surface Model, and (3) the Top of Overlay Model furnished by the Engineer, the Contractor shall be responsible for all calculations, surveying, and measuring required for setting, maintaining, and resetting equipment and materials necessary for the construction of the overlay to the final grade profile. The Engineer may post-check the Contractor's surveying, but these post-checks shall not relieve the Contractor of responsibility for internal survey quality control.

6-21.3(5) Operations on the Bridge Deck

The following apply to all Contractor operations on the bridge deck, including but not limited to Scarification, cleaning concrete surfaces, Type 1 and Type 2 Deck Repair, sandblasting, water-blasting, placing, consolidating, finishing, curing, sawing, and sealing concrete:

1. The Contractor shall protect adjacent traffic from flying debris in accordance with its Debris containment and disposal plan submitted in accordance with Section 6-21.3(3).
2. The Contractor shall collect, contain, and dispose of all concrete debris in accordance with its Debris Containment and Disposal Plan submitted in accordance with Section 6-21.3(3).
3. The Contractor shall collect, contain, and dispose all Process Wastewater in accordance with Sections 5-01.3(11), 6-21.3(6)D, and the Contractor's Process Wastewater Containment and Disposal Plan submitted in accordance with Section 6-21.3(3). The Contractor shall comply with applicable regulations concerning such water disposal. Process Wastewater shall not be in-filtrated.

6-21.3(6) Scarifying Concrete Surface

6-21.3(6)A Prerequisites to Scarification

Scarifying shall not begin until after the Contractor has completed all the following:

1. Demonstrated that all Work needed to complete items 3, 4, 5, 6, 7, and 8 of Section 6-21.3(1) can and will be completed in one and only one construction season.
2. Submitted all submittals required in Section 6-21.3(3) and addressed all the Engineer's comments to the satisfaction of the Engineer.
3. Has received the Idealized Existing Bridge Deck Surface Model and Top of Overlay Model from the Engineer.

6-21.3(6)B Restrictions on Scarification Methods

The method used to Scarify the bridge deck from the Existing Bridge Deck Surface down to ½ inch above the Scarification Depth shall be one of the following:

1. Rotary Milling
2. Hydro-Demolition
3. Power Driven Hand Tools

The method used to Scarify the bottom ½ inch of Scarification Depth shall be one of the following:

1. Hydro-Demolition with rotary heads
2. Power Driven Hand Tools

The method used for Scarification within 12 inches, horizontally, of existing: bridge expansion joint headers, steel expansion joint assemblies, concrete butt joints between back of pavement seats and bridge approach slabs, bridge drain assemblies, and thrie-beam post steel anchorage assemblies fastened to the side or top of the Structure Deck shall be one of the following:

1. Hydro-Demolition
2. Power Driven Hand Tools

All areas of the deck that are inaccessible to the selected Scarifying machine shall be Scarified to remove the concrete surface matrix by a method acceptable to the Engineer.

6-21.3(6)C Scarification Requirements

Prior to Scarification with a rotary milling machine, the Contractor shall verify the depth of the rebar. The Contractor shall conduct rotary milling in such a manner as to avoid snagging rebar.

The Contractor shall Scarify the Existing Bridge Deck to the Scarification Depth and horizontal limits shown in the Plans. Determination of the elevation of the bottom of the Scarification Depth shall be in accordance with the definition of Scarification Depth in Section 6-21.1(1).

Dense, sound areas of existing bridge deck concrete that are difficult to remove with hydro-demolition or rotary milling equipment may, with the concurrence of the Engineer, be left in place provided the concrete is Scarified to a depth sufficient to provide a minimum thickness of Modified Concrete Overlay of 1.0 inch when measured from the peaks of the Scarified surface to the Top of Overlay Model.

6-21.3(6)D Requirements for Rotary Milling

The requirements of this section are in addition to the requirements for rotary milling equipment in Section 6-21.3(2)A.

Rotary milling machines shall be operated to Scarify no deeper than the criteria in (1) and (2) below; whichever criteria limits use to the shallowest depth of cut shall control:

1. ½ inch above the Scarification Depth, measured from the Scarification Depth to the deepest cuts of a tooth, or
2. 1.0 inch above the top of rebar, measured between the deepest cuts of a tooth to the top of the shallowest rebar.

Operation of the rotary milling machine shall be continuously monitored to ensure compliance with the requirements of (1) and (2) above.

If the rotary milling machine contacts the structure deck in excess of the Scarification Depth tolerances provided in Section 6-21.3(2)A, or Scarifies deeper than the limitations on depth of cut in this section, or contacts steel reinforcing bars at any time, the Contractor shall immediately cease rotary milling and notify the Engineer. Rotary milling shall not resume until completion of the appropriate adjustments to the rotary milling machine and receiving the Engineer's concurrence to resume.

6-21.3(6)E Requirements for Hydro-Demolishing

The requirements of this section are in addition to the requirements for hydro-demolishing equipment in Section 6-21.3(2)B.

All hydro-demolition that is intended to remove concrete within a ½ inch immediately above the Scarification Depth shall be performed with rotary heads.

The depth of concrete removal with the hydro-demolition machine shall not exceed 2.0 inches in sound concrete in one pass. If the Scarification Depth is less than or equal to 2.0 inches, the hydro-demolition machine shall remove sound concrete to the Scarification Depth in one pass of the machine, within a tolerance of ±¼ inch. Because this pass includes removal of the ½ inch immediately above the Scarification Depth, the full depth of removal shall be performed with rotary heads.

If the Scarification Depth is greater than 2.0 inches, multiple passes shall be required, with no pass removing greater than 2 inches of sound concrete.

Once the operating parameters of the Hydro-Demolition machine are defined by programming and calibration as specified in Section 6-21.3(2)B, they shall be monitored every 20 feet as the machine progresses across the bridge deck and adjusted as needed to prevent the unnecessary removal of sound concrete below the required Scarification Depth.

The Contractor shall maintain a minimum production rate of 250-square feet per hour during the deck Scarifying process.

All water used in the Hydro-Demolition process shall be potable. Stream or lake water will not be permitted.

All bridge drains and other outlets within 100 feet of the Hydro-Demolition machine shall be temporarily plugged during the Hydro-Demolition operation. When Scarifying a bridge deck passing over traffic lanes, the Contractor shall protect the traffic below by restricting and containing Scarifying operations and implementing traffic control measures.

6-21.3(6)F Scarification Cleanup

After Scarifying is complete to the Scarification Depth and all concrete debris and rubble have been removed - over the entire lane or strip to be overlaid in one operation – the Contractor shall thoroughly vacuum and wash with high-pressure water the entire surface of all dust, freestanding water, and loose particles, to clean it in preparation for ASTM D4580 Method B inspection.

6-21.3(7) Deck Repair

6-21.3(7)A Classification

Deck repair will be classified as Type 1 Deck Repair or Type 2 Deck Repair. The determination of whether an area will be classified as Type 1 or Type 2 will not be made until after completion of deck repair excavation, repair of steel reinforcing bars, and removal of concrete debris.

6-21.3(7)B Chain Drag

After the entire lane or strip to be overlaid has been Scarified and cleaned as required in Section 6-21.3(6), the entire surface shall be inspected by the Contractor, in the presence of the Engineer, in accordance with ASTM D4580, Method B. Based on that inspection, the Contractor shall mark those areas, meeting any of the following criteria, for removal:

1. Unsound concrete in accordance with ASTM D4580, Method B.
2. Lack of bond between existing concrete and reinforcing steel.
3. All existing nonconcrete patches.

After all deck repair excavation is complete, the Contractor shall measure and submit to the Engineer as a Type 1 Working Drawing the location and size of each area identified above by station, offset, length, width, average depth, and deck repair Type, using the form provided by the Engineer.

6-21.3(7)C Deck Repair Excavation

The areas marked for removal in Section 6-21.3(7)B shall be excavated with equipment as described in Section 6-21.3(2)C, or hydro-demolishing equipment. Excavation shall be to the depth necessary to remove all loose and unsound material, without damaging reinforcing steel or sound concrete.

Care shall be taken in removing the deteriorated material to not damage the existing sound concrete or steel reinforcing bars that are to remain in place. All removal shall be accomplished by making vertical edges at the boundaries of the repair area. In no case shall the depth of a sawn vertical cut exceed ¾ inch or to the top of the top steel reinforcing bars, whichever is less.

Bridge deck areas outside the repair area damaged by the Contractor's operations shall be repaired by the Contractor at no additional expense to the Contracting Agency, and to the satisfaction of the Engineer.

6-21.3(7)D Repair of Steel Reinforcing Bars

Where existing steel reinforcing bars inside deck repair areas show natural deterioration greater than 20-percent section loss, the Contractor shall furnish and place steel reinforcing bars alongside the deteriorated bars in accordance with the details shown in the Standard Plans. Payment for such extra Work will be by force account as provided in Section 1-09.6.

All reinforcing steel damaged due to the Contractor's operations shall be repaired by the Contractor. Damage to rebar shall be understood to include damage to epoxy coating.

The repair shall be as follows or as directed by the Engineer:

1. Damage to epoxy coating, when present on existing steel reinforcing bars, shall be repaired in accordance with Section 6-02.3(24)H.
2. Damage to bars resulting in a section loss of 20 percent or more of the bar area shall be repaired by chipping out the adjacent concrete and splicing a new bar of the same size. Concrete shall be removed to provide a ¾-inch minimum clearance around the bars. The splice bars shall extend a minimum of 40 bar diameters beyond each end of the damage.

3. All bars partially or completely removed from the deck shall have the damaged portions removed and spliced with new bars as outlined in item 2 above.

For bridge decks constructed under the same contract as the concrete overlay, all reinforcing steel damaged due to the Contractor's operations shall be repaired by the Contractor at no additional expense to the Contracting Agency.

For bridge decks not constructed under the same Contract as the modified concrete overlay, responsibility for costs to repair damage shall be allocated as follows:

1. Repairing damage that occurs during hydro-demolition to coatings on existing reinforcing steel that is at or above the Scarification Depth shall be paid for in accordance with Section 1-09.6.
2. Repairing damage to existing reinforcing steel that is below the Scarification Depth shall be at no additional expense to the Contracting Agency.

6-21.3(7)E Type 1 Deck Repair

An area will be classified as a Type 1 Deck Repair when the completed concrete excavation complies with both of the following:

1. Concrete excavation extends below the Scarification Depth and
2. Concrete excavation either (a) exposes no more than one-half the periphery of a bottom bar of the top layer of steel reinforcement, or (b) the length of an exposed bar does not exceed 12-continuous inches along the length of the bar.

However, Type 1 Deck Repair does not include concrete excavation which occurred during Scarification.

The scope of Work for Type 1 Deck Repair includes:

1. Excavating and disposing of the unsound concrete within the repair area, below the Scarification Depth.
2. Repair of steel reinforcing bars damaged by the Contractor.
3. Sandblast or water-blast the finished surface and exposed rebar (may be done during Surface Preparation as described in Section 6-21.3(8)A).
4. Flush, saturate, and remove freestanding water.

Type 1 Deck Repair does not include filling the excavated area with Modified Concrete (or other concrete).

If the modified concrete overlay is placed on a bridge deck as part of the same Contract as the bridge deck construction, then all Work associated with Type 1 Deck Repair shall be performed at no additional expense to the Contracting Agency.

6-21.3(7)F Type 2 Deck Repair

An area will be classified as a Type 2 Deck Repair when the completed concrete excavation complies with both of the following:

1. Concrete excavation took place below the Scarification Depth and
2. Concrete excavation either exposes more than one-half the periphery of a bottom bar of the top layer of steel reinforcement or the length of an exposed bar exceeds 12-continuous inches along the length of the bar.

However, Type 2 Deck Repair does not include concrete excavation which occurred during Scarification.

The scope of Work for Type 2 Deck Repair includes:

1. Excavating and disposing of the unsound concrete within the repair area, below the Scarification Depth.
2. Repairing steel reinforcing bars damaged by the Contractor.

3. Sandblasting or water-blasting the area and exposed rebar prior to placing patching concrete.
4. Flushing, saturating, and removing freestanding water.
5. All work related to patching and curing the excavated area with Concrete Class M or modified concrete.

If the modified concrete overlay is placed on a bridge deck as part of the same Contract as the bridge deck construction, then all Work associated with Type 2 Deck Repair shall be performed at no additional expense to the Contracting Agency.

6-21.3(7)G Filling and Curing Deck Repair Areas

Type 1 Deck Repairs shall be filled with Modified Concrete as part of and during placement of the Modified Concrete Overlay. See Section 6-21.3(8).

Type 2 Deck Repairs shall be patched with either concrete Class M or modified concrete. The top of these patches shall be at the bottom elevation of the Scarification Depth. Regardless of whether the Contractor uses concrete Class M or modified concrete for the patching material, all Type 2 deck repair patching shall be performed well enough in advance of the modified concrete overlay to allow all patches to cure as required below.

Before placing Concrete Class M or modified concrete in the Type 2 deck repairs, the Contractor shall clean the surfaces to which the concrete will be applied (including rebar) by (1) water-blasting, or (2) sandblasting and flushing with water. The Contractor shall make sure the existing concrete is well saturated at the time of placing concrete in the Type 2 deck repairs but shall remove all freestanding water prior to placing the concrete. The Contractor shall place concrete in the Type 2 deck repair areas while the existing concrete is wet. It shall be consolidated in accordance with Section 6-02.3(8) and wet cured for a minimum of 42 hours in accordance with the same requirements as the Modified Concrete Overlay in Section 6-21.3(10), but separately and prior to placing the Modified Concrete Overlay. During the curing period of concrete placed in Type 2 deck repairs, all vehicular and foot traffic shall be prohibited in the repair area.

6-21.3(8) Modified Concrete Overlay

6-21.3(8)A Surface Preparation

When modified concrete is to be placed against the concrete in a previously placed transverse joint, lane, or strip, the previously placed concrete shall be sawed back 6 inches to straight and vertical edges and shall be sandblasted or water-blasted before new concrete is placed. The Engineer may decrease the 6-inch saw back requirement to 2 inches minimum, if a bulkhead was used during previous concrete placement and the concrete was hand vibrated along the bulkhead.

Following the completion of all Type 1 and Type 2 Deck Repairs (including placement and curing of patching Type 2 Deck Repair areas), the entire lane or strip being overlaid shall undergo final cleaning. Final cleaning shall be accomplished in one shift and consists of the following, in the sequence shown:

1. Remove oil, grease, rust and other foreign material that may still be present. These materials shall be removed by detergent-cleaning or other method accepted by the Engineer.
2. Sandblast the area cleaned in item 1.
3. Remove all blasting media, films, laitance, and other debris on the entire lane or strip to be overlaid by water-blasting the entire lane or strip to be overlaid.
4. Remove all standing water by vacuuming.
5. Remove remaining films, laitance, foreign material, or other debris.

If, after final cleaning, the lane or strip being overlaid becomes contaminated - by runoff, concrete laitance, or other debris - the Contractor shall repeat the sequence in items 1 through 5.

Concrete placement shall begin within 24 hours of the completion of final cleaning for the portion of the deck to be overlaid. If concrete placement has not begun within 24 hours of final cleaning, the lane or strip being overlaid shall be cleaned by a light sandblasting followed by washing with the high-pressure water spray or by cleaning with the high-pressure spray within 24 hours of overlay placement.

6-21.3(8)B Restrictions on Other Work

Hand tool chipping, sandblasting, water-blasting, and cleaning in areas adjacent to a lane or strip being cleaned in final preparation for placing modified concrete shall be discontinued when final preparation is begun. Scarifying and hand tool chipping shall remain suspended until the concrete has been placed and the requirement for curing time has been satisfied. Sandblasting and cleaning shall remain suspended for the first 24 hours of curing time after the completion of concrete placing.

Scarification and debris removal may proceed during the final cleaning and overlay placement phases of the Work on adjacent portions of the Structure so long as the Scarification and debris removal operations are confined to areas which are a minimum of 100 feet away from the defined limits of the final cleaning or overlay placement in progress. If the Scarification and debris removal impedes or interferes in any way with the final cleaning or overlay placement as determined by the Engineer, the Scarification and debris removal Work shall be terminated immediately and the Scarification and debris removal equipment removed sufficiently away from the area being prepared or overlaid to eliminate the conflict. If the grade is such that water and contaminants from the Scarification and debris removal operation will flow into the area being prepared or overlaid, the Scarification and debris removal operation shall be terminated and shall remain suspended for the first 24 hours of curing time after the completion of concrete placement.

Traffic other than required construction equipment will not be permitted on any portion of the lane or strip being overlaid that has undergone final preparation for placing concrete unless allowed by the Engineer. To prevent contamination, all equipment allowed on the deck after final cleaning shall be equipped with drip guards.

6-21.3(8)C Finishing Machine Setup

Screed rails upon which the finishing machine travels shall be placed outside the area to be overlaid. Interlocking rail sections or other approved methods of providing rail continuity are required.

Hold-down devices shot into the concrete are not permitted unless the concrete is to be subsequently overlaid. Hold-down devices of other types leaving holes in the exposed area will be allowed provided the holes are subsequently filled with mortar conforming to Section 9-20.4(2) mixed at a 1:2 cement/aggregate ratio. Hold-down devices shall not penetrate the existing deck by more than ¾ inch.

A dry-run of the screed shall demonstrate a finished surface that is + ¼ inch/- ⅛ inch from the Engineer's Top of Overlay Model. The finishing machine shall be passed over the entire surface to be overlaid and the final screed rail adjustments shall be made to comply with these tolerance limits. After adjustments are made to demonstrate compliance with these tolerance limits, changes in the finishing machine elevation controls will not be allowed. The Contractor shall be responsible for setting screed control to obtain the specified Top of Overlay Model as well as the finished surface smoothness requirements specified in Section 6-02.3(10)D3.

6-21.3(8)D Placing Modified Concrete Overlay

6-21.3(8)D1 Pre-Overlay Conference

Five to ten working days prior to modified concrete overlay placement, a pre-overlay conference shall be held to discuss equipment, construction procedures, personnel, and previous results. Inspection procedures shall also be reviewed to ensure coordination. Those attending representing the Contractor shall include the superintendent and all foremen in charge of placing, finishing, and curing the modified concrete overlay.

If the project includes more than one bridge deck, an additional conference shall be held just before placing modified concrete overlay for each subsequent bridge deck.

6-21.3(8)D2 Placement of Modified Concrete Overlay

The Contractor shall not begin placing the modified concrete overlay until the Engineer agrees that:

1. The Contractor has complied with the requirements of Sections 6-21.3(8)A, 6-21.3(8)B, 6-21.3(8)C, and the 6-hour pre-saturation requirement in this section.
2. Modified concrete overlay producing and placement rates will be high enough to meet placing and finishing deadlines,
3. Finishers with enough experience have been employed, and
4. Adequate finishing tools and equipment are at the site.
5. Rain is not expected.
6. Concrete shall not be placed when the temperature of the existing concrete surface is less than 45°F or greater than 75°F, and wind velocity one foot above the bridge deck is in excess of 10 mph. If the Contractor elects to Work at night to meet these criteria, adequate lighting shall be provided at no additional expense to the Contracting Agency.

Concrete placement shall be made in accordance with Section 6-02 and the following requirements:

1. Concrete shall not be placed against the edge of an adjacent lane or strip that is less than 36 hours old.
2. After the lane or strip to be overlaid has been prepared, it shall be thoroughly saturated and kept continuously wet with water (pre-saturated) for a minimum period of 6 hours prior to placement of the concrete. All freestanding water shall be removed prior to concrete placement. During concrete placement, the lane or strip shall be kept moist.
3. Immediately ahead of placement of the modified concrete, a slurry of the modified concrete, excluding coarse aggregate, shall be thoroughly brushed into the surface prior to the overlay placement. Care shall be exercised to ensure that the surface receives a thorough, even coating of slurry and that the rate of progress is limited so that the brushed concrete does not become dry before it is covered with additional concrete as required for the final grade. All aggregate which is segregated from the mix during the brushing operation shall be removed from the deck and disposed of by the Contractor.
4. The concrete shall then be promptly and continuously delivered and deposited on the placement side of the finishing machine.
5. The Contractor shall ensure that a sufficient number of trucks are used for concrete delivery to obtain a consistent and continuous delivery and placement of concrete throughout the concrete placement operation.

6. Adequate precautions shall be taken to protect freshly placed concrete in the event that rain begins during placement. Concrete that is damaged by rain shall be removed and replaced by the Contractor at no additional expense to the Contracting Agency, and to the satisfaction of the Engineer.
7. Concrete shall not be placed when the temperature of the existing concrete surface is less than 45°F or greater than 75°F, and wind velocity one foot above the bridge deck is in excess of 10 mph. If the Contractor elects to Work at night to meet these criteria, adequate lighting shall be provided at no additional expense to the Contracting Agency.
8. If concrete placement is stopped for a period of ½ hour or more, the Contractor shall install a bulkhead transverse to the direction of placement at a position where the overlay can be finished full width up to the bulkhead. The bulkhead shall be full depth of the overlay and shall be installed to grade. The concrete shall be finished and cured in accordance with these Specifications.
9. Further placement is permitted after the bulkhead is removed, the previously placed concrete is sawn back from the bulkhead to a point designated by the Engineer to straight and vertical edges, all previously placed slurry scrub is completely removed, the Contractor has performed surface preparation to all contaminated areas in accordance with 6-21.3(8)A including the sawn vertical edge, and one of the following waiting periods:
    a. After a minimum of 12 hours of curing (in accordance with Section 6-21.3(10)) the concrete edge that was bulkheaded, or
    b. In less than 12 hours of curing (in accordance with Section 6-21.3(10)) the concrete edge that was bulkheaded provided a gap is left in the lane or strip of sufficient width for the finishing machine to clear the transverse bulkhead installed where concrete placement was stopped.

6-21.3(8)E Acceptance Testing Modified Concrete

When the Contractor has released the Concrete for acceptance testing, the Engineer will perform slump, temperature, and entrained air tests in accordance with Section 6-02.3(5)D and as specified in this section. Concrete samples for testing shall be supplied to the Engineer in accordance with Section 6-02.3(5)E. Concrete from the first truckload shall not be placed on the deck until tests for acceptance have been completed by the Engineer and the results indicate that the concrete is within acceptable limits. Sampling and testing will continue for each load until two successive loads meet all applicable acceptance test requirements. Except for the first load of concrete, up to ½ cubic yard may be placed on the deck prior to testing for acceptance. After two successive tests indicate that the concrete is within specified limits, the sampling and testing frequency may decrease to one for every three truckloads. Loads to be sampled will be selected in accordance with the random selection process outlined in FOP for WAQTC TM2.

When the results of subsequent acceptance test indicates that the concrete does not conform to the specified limits, the sampling and testing frequency will be resumed for each truckload. Whenever two successive subsequent tests indicate that the concrete is within the specified limits, the random sampling and testing frequency of one for every three truck loads may resume.

6-21.3(9) Finishing Modified Concrete Overlay

Finishing shall be accomplished in accordance with the applicable portions of Section 6-02.3(10), Section 6-21.3(2)G, and as follows. Concrete shall be placed and struck-off approximately ½ inch above final grade and then consolidated and finished to final grade with a single pass (the Engineer may require additional passes) of the finishing machine. Hand finishing may be necessary to close up or seal off the surface. The final product shall be a dense uniform surface.

Construction dams shall be separated from the newly placed concrete by passing a pointing trowel along the inside surfaces of the dams. Care shall be exercised to ensure that this trowel cut is made for the entire depth and length of the dams after the concrete has stiffened sufficiently that it does not flow back.

Deck finishing machine screed rails may be removed at any time after the modified concrete overlay has taken an initial set. Adequate precautions shall be taken during the removal of the finishing machine and rails to protect the edges of the new surfaces.

6-21.3(10) Curing Modified Concrete Overlay

As the finishing operation progresses, the concrete shall be immediately covered with a single layer of clean, new or used, wet burlap. The burlap shall have a maximum width of 6 feet. The Engineer will determine the suitability of the burlap for reuse, based on the cleanliness and absorption ability of the burlap. Care shall be exercised to ensure that the burlap is well drained and laid flat with no wrinkles on the deck surface. Adjacent strips of burlap shall have a minimum overlap of 6 inches.

Once in place the burlap shall be lightly fog sprayed with water. A separate layer of white, reflective type polyethylene sheeting shall immediately be placed over the wet burlap.

As an alternative to the application of burlap and fog spraying described above, the Contractor may propose a curing system using proprietary curing blankets specifically manufactured for bridge deck curing. The Contractor shall submit a Type 2 Working Drawing consisting of details of the proprietary curing blanket system, including product literature and details of how the system is to be installed and maintained.

The burlap shall be kept wet continuously and the wet curing regimen as described shall remain in place for a minimum of 42-hours.

6-21.3(11) Checking for Bond

After the requirements for curing have been met, the entire overlaid surface shall be sounded by the Contractor in accordance with ASTM D4580, in a manner accepted by and in the presence of the Engineer, to ensure total bond of the concrete to the bridge deck.

6-21.3(11)A Repairing Unbonded Concrete

All Work in this Section, 6-21.3(11)A, shall be performed at no additional expense to the Contracting Agency.

Concrete in unbonded areas shall be removed and replaced in accordance with the following:

1. Section 6-21.3(7)C, Deck Repair Excavation, and the following: Saw cut the edges around the repair area with an edge perpendicular to the concrete surface. The depth of the saw cut shall be ¾ inch but shall be reduced if necessary to avoid damaging the reinforcement. The geometry of the repair perimeter shall be rectangular, avoiding reentrant corners. All concrete in the removal area shall be removed by chipping, or other approved mechanical means to the depth necessary to remove all loose or unsound concrete. All removal shall maintain square edges at the boundaries of the repair area.
2. Section 6-21.3(7)D, Repair of Steel Reinforcing Bars
3. Section 6-21.3(8)A, Surface Preparation
4. Section 6-21.3(8)B, Restrictions on Other Work
5. Section 6-21.3(8)D, Placing Modified Concrete Overlay. Concrete for patching shall be the same modified concrete as used in the overlay.
6. Section 6-21.3(10), Curing Modified Concrete Overlay.

After curing requirements have been met, all repaired areas shall be sounded by the Contractor in accordance with this section, and again repaired if any unsound or unbonded concrete is found.

6-21.3(12) Sealing and Texturing Concrete Overlay

After the requirements for checking for bond have been met, all cold joints (transverse and longitudinal, and around repair areas) and visible cracks shall be filled and sealed with a high molecular weight methacrylate resin (HMWM). Cracks 1⁄16 inch and greater in width shall receive two applications of HMWM. Immediately following the application of HMWM, the wetted surface shall be coated with sand for abrasive finish.

After all cracks and cold joints have been filled and sealed and the HMWM resin has cured, the concrete overlay surface shall receive a longitudinally sawn texture in accordance with Section 6-02.3(10)D5.

Traffic shall not be permitted on the finished concrete until it has reached a minimum compressive strength of 3,000 psi as verified by rebound number determined in accordance with ASTM C805 and the longitudinally sawn texture is completed.

6-21.4 Measurement

Scarifying concrete surface will be measured by the square yard of surface Scarified.

Type 1 Deck Repair and Type 2 Deck Repair will be measured by the square foot of surface area, below the Scarification Depth, of deck concrete removed in accordance with Section 6-21.3(7). Determination of whether a deck repair is Type 1 or Type 2 shall be in accordance with Section 6-21.3(7).

Modified concrete overlay will be measured by the cubic foot of material placed. The volume will be determined from the concrete supplier’s Certificate of Compliance for each batch delivered less waste. Waste is defined as the following:

1. Material not placed.
2. Material placed in excess of 6 inches outside a longitudinal joint or transverse joint.

Finishing and curing modified concrete overlay will be measured by the square yard of overlay surface finished.

6-21.5 Payment

Payment will be made for each of the following Bid items that are included in the Proposal:

“Surveying for Modified Conc. Overlay”, lump sum.

The lump sum contract price for “Surveying for Modified Conc. Overlay” shall be full pay to perform the work as specified, including establishing secondary survey control points, performing survey quality control, creating the Existing Bridge Deck Surface Model, and recording, compiling, and submitting the survey records to the Engineer.

“Scarifying Conc. Surface”, per square yard.

The unit Contract price per square yard for “Scarifying Conc. Surface” shall be full pay for performing the Work as specified, including: testing and calibration of the machines and tools used; Scarifying to the Scarification Depth; excavation of unsound concrete below the Scarification Depth when such unsound concrete is removed during the pass of the hydro-demolition machine intended to stop at the Scarification Depth; disposal of all debris generated; scarification cleanup.

"Type 1 Deck Repair", per square foot.

The unit contract price per square foot for Type 1 Deck Repair shall be full pay for performing the work as specified, including excavating and disposing concrete, and repair of concrete or rebar damaged by the Contractor's operations.

"Type 2 Deck Repair", per square foot.

The unit contract price per square foot for Type 2 Deck Repair shall be full pay for performing the work as specified, including: excavating and disposing concrete; sandblasting and water-blasting; placing, consolidating, finishing, and curing concrete patches in Type 2 deck repairs; repair of concrete or rebar damaged by the Contractor's operations.

"Modified Conc. Overlay", per cubic foot.

The unit contract price per cubic foot for "Modified Conc. Overlay" shall be full pay for furnishing and placing the modified concrete overlay, including the overlay material placed into Type 1 Deck Repairs in accordance with Section 6-21.3(7)G, and preparing the surface for the overlay including cleaning, sandblasting, and water-blasting.

"Finishing and Curing Modified Conc. Overlay", per square yard.

The unit Contract price per square yard for "Finishing and Curing Modified Conc. Overlay" shall be full pay for performing all work required under Sections 6-21.3(9), 6-21.3(10), 6-21.3(11), and 6-21.3(12) including finishing the modified concrete overlay, curing the modified concrete overlay, checking for bond, repairing unbonded areas, sealing all cracks and cold joints, sanding, and providing longitudinally sawn texture.

Payment for the following shall be considered incidental to and included in the unit contract items included in the contract:

1. All Work and related costs for implementing the Process Wastewater Containment and Disposal Plan.
2. All Work and related costs for implementing the Debris Containment and Disposal plan.
3. All Work and related costs for protecting adjacent traffic from flying debris.

6-22 Modified Concrete Overlay - Latex

6-22.1 Description

This Section 6-22 is for Contracts with the bid item “Modified Concrete Overlay” when the Contractor elects to use latex modified concrete for the overlay. If the Contractor elects to use microsilica modified concrete or fly ash modified concrete for the overlay, use Section 6-21, Modified Concrete Overlay – Microsilica or Fly Ash.

This Work consists of surveying, Scarifying the existing concrete bridge deck, removing and replacing unsound concrete below the Scarification Depth, and placing, finishing, curing, and texturing a modified concrete overlay using latex modified concrete.

6-22.1(1) Definitions

The Contractor shall comply with 6-21.1(1).

6-22.2 Materials

Materials shall meet the requirements of the following sections:

High Molecular Weight Methacrylate (HMWM) Resin for Crack and Cold Joint Sealing	6-21.2(3)
Sand for Abrasive Finish of Cracks and Cold Joints	6-21.2(3)
Portland Cement	9-01.2(1)
Blended Hydraulic Cement	9-01.2(1)B
Latex Admixture	6-22.2(1)A1
Fine Aggregate	9-03.1
Course Aggregate	9-03.1
Mortar	9-20.4
Burlap Cloth	9-23.5
Admixtures	9-23.6
Water for Concrete	9-25.1

6-22.2(1) Modified Concrete

6-22.2(1)A Latex Modified Concrete

The modified concrete shall be latex modified concrete. Latex modified concrete shall be a workable mix, uniform in composition and consistency. Mix proportions per cubic yard shall be as follows:

Type I or II Portland Cement, or Type IL(X) Blended hydraulic Cement	1.00 parts by weight
Fine Aggregate	2.40 to 2.75 parts by weight
Coarse Aggregate	1.75 to 2.00 parts by weight
Latex Admixture	3.50-gallons per bag of cement
Water/Cement Ratio	0.33 maximum
Air Content of Plastic Mix	6 percent maximum
Slump	7 inches maximum

The aggregates shall be proportioned such that the amount of aggregate passing the No. 4 sieve is 65 ± 5 percent of the total aggregate (fine plus coarse). All calculations shall be based on dry weights.

The moisture content of the fine aggregate and coarse aggregate shall be no more than 3.0 and 1.0 percent, respectively, above the saturated surface dry condition.

The maximum allowable and actual water/cementitious ratio shall be calculated using all of the available mix water, including water added at the plant, water added in transit and at the job site, water in all admixtures, 52 percent of the latex admixture, and the free water in the aggregates but not the water absorbed by the aggregates. The following are considered cementitious materials: portland cement and blended hydraulic cement.

Ground granulated blast furnace slag, fly ash, microsilica fume, and Type 3 portland cement shall not be used in Latex Modified concrete.

Fine aggregate shall be Class 1. Coarse aggregate shall be AASHTO grading No. 7 or No. 8.

The Contractor shall adjust the slump to accommodate the gradient of the bridge deck, subject to the maximum slump specified.

6-22.2(1)A1 Latex Admixture

Latex admixture shall be a non-toxic, film-forming, polymeric emulsion in water to which all stabilizers have been added at the point of manufacture. The latex admixture shall be homogeneous and uniform in composition, and shall conform to the following:

Polymer Type	Styrene Butadiene
Stabilizers:	
Latex	Non-ionic surfactants
Portland Cement	Polydimethyl siloxane
Percent Solids	46.0 to 49.0
Weight per Gallon	8.4 pounds at 77 °F
Color White PH (as shipped)	9 minimum
Freeze/Thaw Stability	5 cycles (5°F to 77°F)
Shelf Life	2 years minimum

Latex admixture will be accepted based on submittal of a Manufacturer's Certificate of Compliance in accordance with Section 1-06.3.

6-22.2(1)A2 Storing and Handling Latex Admixture

Latex admixture shall be kept in suitable containers that will protect it from freezing and from exposure to temperatures in excess of 85°F. Containers of the admixture shall not be stored in direct sunlight for periods in excess of 10 days. When stored in direct sunlight the top and sides of the containers shall be covered with insulating blanket material.

Storage of the latex admixture may extend over a period greater than 10 days as long as the conditions specified above are maintained and the latex admixture is agitated or stirred once every 10 days. Stirring or agitation of the admixture shall be done mechanically in accordance with the manufacturer's recommendation. If the ambient temperature is higher than 85°F at any time during the storage period, the admixture shall be covered by insulated blankets or other means that will maintain the admixture temperature below 85°F.

Latex admixture shall be strained through a Number 10 strainer at the time it is introduced into the mixing tank from the storage containers.

6-22.2(2) Concrete Class M

The Contractor shall comply with Section 6-21.2(2).

6-22.2(3) Materials for Sealing Cracks and Cold Joints

The Contractor shall comply with Section 6-21.2(3).

6-22.3 Construction Requirements

6-22.3(1) Sequence of Work

The Contractor shall comply with Section 6-21.3(1).

6-22.3(2) Equipment

6-22.3(2)A Rotary Milling Machines

The Contractor shall comply with Section 6-21.3(2)A.

6-22.3(2)B Hydro-Demolition Machines

The Contractor shall comply with Section 6-21.3(2)B.

6-22.3(2)C Power Driven Hand Tools

The Contractor shall comply with Section 6-21.3(2)C.

6-22.3(2)D Air Compressor

The Contractor shall comply with Section 6-21.3(2)D.

6-22.3(2)E Vacuum Machine

The Contractor shall comply with Section 6-21.3(2)E.

6-22.3(2)F Water Delivery System

The Contractor shall comply with Section 6-21.3(2)F.

6-22.3(2)G Finishing Machine

The Contractor shall comply with Section 6-21.3(2)G.

6-22.3(2)H Mobile Mixer for Latex Modified Concrete

Latex modified concrete shall be mixed and transported in self-contained, self-propelled, continuous-mixing units conforming to the following requirements:

1. The mixer shall be equipped so that it can be grounded.
2. The mixer shall be equipped to provide positive measurement of the portland cement being introduced into the mix. A recording meter, visible at all times and equipped with a ticket printout, shall be used.
3. The mixer shall be equipped to provide positive control of the flow of water and latex admixture into the mixing chamber. Water flow shall be indicated by a flow meter with a minimum readability of ½ gallon per minute, accurate to ± 1 percent. The water system shall have a bypass valve capable of completely diverting the flow of water. Latex flow shall also be indicated by a flow meter with a minimum readability of 2 gallons per minute, accurate to ± 1 percent. The latex system shall be equipped with a bypass valve suitable for obtaining a calibrated sample of admixture.
4. The mixer shall automatically proportion and blend all components of the specified mix on a continuous or intermittent basis as required by the finishing operation.
5. The mixer shall discharge mixed material through a conventional chute directly in front of the finishing machine.

Inspection of each mobile mixer shall be done by the Contractor in the presence of the Engineer and in accordance with the following requirements:

1. Check the manufacturer's inspection plate or mix setting chart for the serial number, the proper operating revolutions per minute (rpm), and the approximate number of counts on the cement meter to deliver 94 pounds of cement.
2. Make a general inspection of the mobile mixer to ensure cleanliness and good maintenance practices.
3. Check to see that the aggregate bins are empty and clean and that the bin vibrators work.
4. Verify that the cement aeration system operates, that the vent is open, and that the mixer is equipped with a grounding strap. Check the cement meter feeder to ensure that all fins and pockets are clean and free from accumulated cement. If the operator cannot demonstrate, through visual inspection, that the cement meter feeder is clean, all cement shall be removed from the bin and the cement meter feeder inspected. The aeration system shall be equipped with a gauge or indicator to verify that the system is operating.
5. Verify that the main belt is clean and free of accumulated materials.
6. Check the latex strainer to ensure cleanliness.

The initial calibration shall be in accordance with items 6-22.3(2)H1 through 6-22.3(2)H5 as follows:

6-22.3(2)H1 Cement Meter

a. Refer to the truck manufacturer's mix setting chart to determine the specified operating rpm and the approximate number of counts required on the cement meter to deliver 94 pounds of cement.
b. Place at least 40 bags (about 4,000 pounds) of cement in the cement bin.
c. Ensure the mixer is resting on a level surface.
d. Ensure the mixer is grounded.
e. Adjust the engine throttle to obtain the specified rpm. Operate the unit, discharging cement until the belt has made one complete revolution. Stop the belt. Reset the cement meter to zero. Position a suitable container to catch the cement and discharge approximately one bag of cement. With a stopwatch, measure the time required to discharge the cement. Record the number of counts on the cement meter and determine the weight of the cement in the container. Repeat the process of discharging approximately one bag of cement until six runs have been made. Reset the cement meter to zero for each run.

Example Calibration Record:

Run No.	Cement Counts	Weight of Cement	Time in Seconds
1	66	95	31
2	68	96	31.2
3	67	95.5	31.0
4	66	95	29.8
5	67	95.25	30.5
6	66	95	30.8
TOTAL	**400**	**571.75**	**184.3**

Pounds of cement per count on cement meter:

$$\frac{\text{Weight of Cement}}{\text{No. of Counts}} = \frac{571.75}{400} = 1.43 \text{ LB./Count}$$

Counts per bag (94 pounds):

$$\frac{94}{1.43} = 65.7 \text{ Counts Bag}$$

Pounds of cement discharged per second:

$$\frac{\text{Weight of Cement}}{\text{Time in Seconds}} = \frac{571.75}{184.3} = 3.10 \text{ LB./SEC.}$$

Required time to discharge one bag:

$$\text{Time} = \frac{94}{3.10} = 30.32 \text{ SEC./Bag}$$

6-22.3(2)H2 Latex Throttling Valve

a. Check to be sure that the latex strainer is unobstructed.

b. The latex throttling valve shall be adjusted to deliver 3.5 gallons of latex (29.4 pounds) for each bag of cement. From the above calculation 30.32 seconds are required to deliver one bag of cement.

c. With the unit operating at the specified rpm, discharge latex into a container for 30.3 seconds and determine the weight of latex. Continue adjusting the valve until 29.4 to 29.5 pounds of latex is discharged in 30.3 seconds. Verify the accuracy of this valve setting three times.

6-22.3(2)H3 Water Flow Meter

a. Set the water flow meter by adjusting it to flow at ½ gallon per minute.

b. Collect and weigh the water discharged during a 1-minute interval with the equipment operating at the specified rpm. Divide the weight of water by 8.34 to determine the number of gallons.

c. Repeat items a. and b., above, with the flow meter adjusted to 1½ gallons per minute.

6-22.3(2)H4 Aggregate Bin Gates

a. Set the gate openings to provide the amount of aggregate required to produce concrete having the specified proportions.

b. Discharge a representative sample of the aggregates through the gates and separate on the No. 4 sieve. Aggregates shall meet the requirements for proportions in accordance with Section 6-22.3(3)E.

c. Adjust the gate openings if necessary to provide the proper ratio of fine aggregate to total aggregate.

6-22.3(2)H5 Mobile Mixer Trial Runs and Calibration

Each mobile mixer shall be operated to produce at least ½ cubic yard of concrete, which shall be in compliance with these Specifications, prior to acceptance of the mobile mixer for job use. The Engineer will perform yield, slump, and air tests on the concrete produced by each mixer. Calibration of each mobile mixer shall be done by the Contractor in the presence of the Engineer. A complete calibration is required on each mixer on each concrete placement unless, after the initial calibration, the personnel having the responsibility of mixer calibration on subsequent concrete placement were present during the initial calibration of the mixer and during the concrete placement operations and are able to verify the dial settings of the initial calibration and concrete placement.

If these criteria are met, a complete calibration need not be repeated provided that a single trial run verifies the previous settings of the cement meter, latex throttling valve, water flow meter, and aggregate gradations, and that the mixer has not left the project and the Engineer is satisfied that a complete calibration is not needed.

6-22.3(2)H6 Production Mixing Latex Modified Concrete

The mobile mixer shall be operated with strict adherence to the procedures set forth by its manufacturer.

A minimum of two mobile mixers will be required at the overlay site for each concrete placement when the total volume of concrete to be placed during the concrete placement exceeds the material storage capacity of a single mobile mixer. Additional mobile mixers may be required if conditions require that material be stockpiled away from the jobsite. The Contractor shall have sufficient mobile mixers on hand to ensure a consistent and continuous delivery and placement of concrete throughout the concrete placement.

Charging the mobile mixer shall be done in the presence of the Engineer. Mixing capabilities shall be such that the finishing operation can proceed at a steady pace.

6-22.3(3) Submittals

The Contractor shall comply with Section 6-21.3(3), with the following revisions:

> Item 5 is revised to read: "5. A Type 2 Working Drawing of the mix designs for concrete class M and Letex Modified Concrete. The Contractor's submittal of a mix design shall be on WSDOT Form 350-040 and shall provide a unique identification for each mix design. A unique identification for the mix design is composed of the combination of the Mix Design Number and the Concrete Plant Number".
>
> Item 8 is added as follows: "8. A Type 1 Working Drawing of samples of the latex admixture and the portland cement (or blended hydraulic cement) for testing and compatibility."

6-22.3(4) Surveying for Modified Concrete Overlay

The Contractor shall comply with Section 6-21.3(4).

6-22.3(5) Operations on the Bridge Deck

The Contractor shall comply with Section 6-21.3(5).

6-22.3(6) Scarifying Concrete Surface

The Contractor shall comply with Section 6-21.3(6).

6-22.3(7) Deck Repair

The Contractor shall comply with Section 6-21.3(7).

6-22.3(8) Modified Concrete Overlay

6-22.3(8)A Surface Preparation

The Contractor shall comply with Section 6-21.3(8)A.

6-22.3(8)B Restrictions on Other Work

The Contractor shall comply with Section 6-21.3(8)B.

6-22.3(8)C Finishing Machine Setup

The Contractor shall comply with Section 6-21.3(8)C.

6-22.3(8)D Placing Modified Concrete Overlay

The Contractor shall comply with Section 6-21.3(8)D.

6-22.3(8)E Acceptance Testing Latex Modified Concrete

The Engineer will perform slump, temperature, and entrained air tests for acceptance in accordance with Section 6-02.3(5)D and as specified in this section after the Contractor has turned over the concrete for acceptance testing. The Engineer will perform testing as the concrete is being placed. Samples shall be taken on the first charge through each mobile mixer and every other charge thereafter. The sample shall be taken after the first 2 minutes of continuous mixer operation. Concrete samples for testing shall be supplied to the Engineer in accordance with Section 6-02.3(5)E.

During the initial proportioning, mixing, placing, and finishing operations, the Engineer may require the presence of a technical representative from the latex admixture manufacturer. The technical representative shall be capable of performing, demonstrating, inspecting, and testing all of the functions required for placement of the latex modified concrete as specified in Section 6-21.3(8)D. This technical representative shall aid in the proper installation of the latex modified concrete. Recommendations made by the technical representative on or off the jobsite shall be adhered to by the Contractor. The Engineer will advise the Contractor in writing a minimum of 5 working days before such services are required.

6-22.3(9) Finishing Modified Concrete Overlay

The Contractor shall comply with Section 6-21.3(9) and the following:

Latex shall not be sprayed on a freshly placed latex modified concrete surface; however, a light fog spray of water is permitted if required for finishing, as determined by the Engineer.

6-22.3(10) Curing Modified Concrete Overlay

The Contractor shall comply with Section 6-21.3(10).

6-22.3(11) Checking for Bond

The Contractor shall comply with Section 6-21.3(11).

6-22.3(12) Sealing and Texturing Concrete Overlay

The Contractor shall comply with Section 6-21.3(12).

6-22.4 Measurement

Scarifying concrete surface will be measured by the square yard of surface Scarified.

Type 1 Deck Repair and Type 2 Deck Repair will be measured by the square foot of surface area, below the Scarification Depth, of deck concrete removed in accordance with Section 6-21.3(7). Determination of whether a deck repair is Type 1 or Type 2 shall be in accordance with section 6-21.3(7).

Modified concrete overlay will be measured by the cubic foot of material placed. For latex modified concrete overlay, the volume will be determined by the theoretical yield of the design mix and documented by the counts of the cement meter less waste. Waste is defined as the following:

1. Material not placed.
2. Material placed in excess of 6 inches outside a longitudinal joint or transverse joint.

Finishing and curing modified concrete overlay will be measured by the square yard of overlay surface finished.

6-22.5 Payment

Payment will be made for each of the following Bid Items that are included in the Bid Proposal:

"Surveying for Modified Conc. Overlay", lump sum.

The lump sum contract price for "Surveying for Modified Conc. Overlay" shall be full pay to perform the work as specified, including establishing secondary survey control points, performing survey quality control, creating the Existing Bridge Deck Surface Model, and recording, compiling, and submitting the survey records to the Engineer.

"Scarifying Conc. Surface", per square yard.

The unit Contract price per square yard for "Scarifying Conc. Surface" shall be full pay for performing the Work as specified, including: testing and calibration of the machines and tools used; Scarifying to the Scarification Depth; excavation of unsound concrete below the Scarification Depth when such unsound concrete is removed during the pass of the hydro-demolition machine intended to stop at the Scarification Depth; disposal of all debris generated; scarification cleanup.

"Type 1 Deck Repair", per square foot.

The unit contract price per square foot for Type 1 Deck Repair shall be full pay for performing the work as specified, including excavating and disposing concrete, and repair of concrete or rebar damaged by the contractor's operations.

"Type 2 Deck Repair", per square foot.

The unit contract price per square foot for Type 2 Deck Repair shall be full pay for performing the work as specified, including: excavating and disposing concrete; sandblasting and water-blasting; placing, consolidating, finishing, and curing concrete patches in Type 2 deck repairs; repair of concrete or rebar damaged by the contractor's operations.

"Modified Conc. Overlay", per cubic foot.

The unit contract price per cubic foot for "Modified Conc. Overlay" shall be full pay for furnishing and placing the modified concrete overlay, including the overlay material placed into Type 1 Deck Repairs in accordance with Section 6-21.3(7)G, and preparing the surface for the overlay including cleaning, sandblasting, and water-blasting.

"Finishing and Curing Modified Conc. Overlay", per square yard.

The unit Contract price per square yard for "Finishing and Curing Modified Conc. Overlay" shall be full pay for performing all work required under Sections 6-22.3(9), 6-22.3(10), 6-22.3(11), and 6-22.3(12) including finishing the modified concrete overlay, curing the modified concrete overlay, checking for bond, repairing unbonded areas, sealing all cracks and cold joints, sanding, and providing longitudinally sawn texture.

Payment for the following shall be considered incidental to and included in the unit contract items included in the contract:

1. All Work and related costs for implementing the Process Wastewater Containment and Disposal plan
2. All Work and related costs for implementing the debris containment and disposal plan.
3. All Work and related costs for protecting adjacent traffic from flying debris.

   The unit Contract price per square yard for "Finishing and Curing Modified Conc. Overlay" shall be full pay for performing all work required under Sections 6-21.3(9), 6-21.3(10), 6-21.3(11), and 6-21.3(12) including finishing the modified concrete overlay, curing the modified concrete overlay, checking for bond, repairing unbonded areas, sealing all cracks and cold joints, sanding, and providing longitudinally sawn texture.

Payment for the following shall be considered incidental to and included in the unit contract items included in the contract:

1. All Work and related costs for implementing the Process Wastewater Containment and Disposal Plan.
2. All Work and related costs for implementing the Debris Containment and Disposal plan.
3. All Work and related costs for protecting adjacent traffic from flying debris.

6-23 Polyester Concrete Overlay

6-23.1 Description

This Work consists of installing polyester concrete bridge deck overlays, preparing the surface of the concrete bridge deck, removing and replacing unsound concrete (deck repair), surveying, and other Work.

6-23.1(1) Definitions

T Existing Bridge Deck Surface - The surface of the existing concrete bridge deck. It follows wheel ruts and other anomalies.

Polyester Concrete Overlay System - All component materials used to complete the system, including the polyester concrete (which is composed of polyester concrete binder and aggregate), primer, initiators, promoters, catalysts, accelerators, inhibitors, sand for abrasive finish, and crack sealing resin. All component materials of the polyester concrete system shall be provided through a single System Provider.

System Provider – The single corporate entity that provides the Polyester Concrete Overlay System that will be installed on this Contract. There shall be only one System Provider.

System Provider Technical Representative - A duly authorized agent of the System Provider, who has the requisite skills and experience.

6-23.1(2) Qualifications

The following shall have the minimum experience as described.

6-23.1(2)A System Provider

The proposed System Provider shall have had direct control and responsibility for the proposed polyester concrete overlay system for the qualifying projects for the overlay system. Qualifying Projects - The Polyester Concrete Overlay System shall have been successfully placed on three overlay projects of similar size and scope to the proposed installation within the past ten years. Previously installed overlay must be in service for a minimum of two years showing no signs of installation deficiency, major distress, excessive wear, non-reflective in-service cracks, insufficient skid resistance, or delamination.

6-23.1(2)B System Provider Technical Representative

The System Provider Technical Representative shall have a minimum of two years of experience with the exact polyester concrete overlay system to be used on this Contract and be completely competent in all aspects of the Work. The Technical Representative shall have experience on a minimum of three successful projects of similar size and scope to the proposed installation. Thin polymer (broadcast) overlay experience will not be accepted.

6-23.1(2)C Polyester Concrete Placement Contractor and Workers

The Contractor that performs the work of placing the polyester concrete system shall have experience on three projects within the past two years placing polyester concrete overlays using equipment as specified herein. Thin polymer (broadcast) overlay experience will not be accepted.

The following employees shall also meet these qualifications:

1. One on-site supervisor.
2. One volumetric mixer operator.
3. One finishing machine operator.

6-23.2 Materials

Materials shall meet the requirements of the following sections:

Polyester Concrete Binder	6-23
Primer	6-23
Aggregate for Polyester Concrete	6-23
Sand for Abrasive Finish	6-23
Crack Sealing Materials	6-23
Portland Cement	9-01.2(1)
Blended Hydraulic Cement	9-01.2(1)B
Fine Aggregate	9-03.1
Coarse Aggregate	9-03.1
Admixtures	9-23.6
Water for Concrete	9-25.1

6-23.2(1) Polyester Concrete System

All components of the polyester concrete system shall be provided by the System Provider.

1. Manufacturer's Certificates of Compliance - The Contractor shall submit a separate Manufacturer's Certificate of Compliance meeting the requirements of Section 1-06.3 for each of the following components of the polyester concrete system: primer, polyester concrete binder, polyester concrete aggregates, polyester concrete, and sand for abrasive finish. Each Manufacturer's Certificate of Compliance shall identify the applicable lot(s) by lot number.
2. Certified Test Results - Each Manufacturer's Certificate of Compliance shall be accompanied by certified test reports from independent labs for all the properties described in Sections 6-23.2(1)A, B, C, D, and E, which are associated with each component. Each certified test report shall identify the lot(s) represented by the test report by lot number.
3. Sampling - The Contracting Agency reserves the right to obtain and test samples of components of the polyester concrete overlay system. This includes requiring submittal of samples prior to the first installation or on-site sampling during construction.

6-23.2(1)A Primer

Primer for the substrate concrete surface shall be a wax-free low odor, high molecular weight methacrylate primer, and consist of a resin, initiator, and promoter. The primer shall conform to the following requirements:

Resin		
Property	**Requirement**	**Test Method**
Viscosity	25 cps maximum (Brookfield RVT with UL adapter, 50 RPM at 77°F)	ASTM D2196
Volatile Content	30% maximum	ASTM D2369
Specific Gravity	0.90 minimum at 77°F	ASTM D1475
Vapor Pressure	1.0 mm Hg, maximum at 77°F	ASTM D 323

Resin with Initiator		
Property	**Requirement**	**Test Method**
Flash Point	180°F minimum	ASTM D 3278

Initiator for the methacrylate resin shall consist of a metal drier and peroxide. If supplied separately from the resin, the metal drier shall not be mixed with the peroxide directly; a VIOLENT EXOTHERMIC REACTION will occur.

6-23.2(1)B Polyester Concrete Binder

Polyester concrete binder shall have the following properties:

1. Be an unsaturated isophthalic polyester-styrene co-polymer.
2. The binder content shall be 12% +/-1% of the weight of the dry aggregate.
3. Be used with a promoter that is compatible with suitable methyl ethyl ketone peroxide and cumene hydroperoxide initiators.
4. Meet the requirements of the following tables.

Resin		
Property	**Requirement**	**Test Method**
Viscosity	75 – 200 cps (RVT No.1 Spindle, 20 RPM at 77°F)	ASTM D2196
Specific Gravity	1.05 to 1.10 at 77°F	ASTM D1475

Resin with Initiator		
Property	**Requirement**	**Test Method**
Contain gamma-methacryloxypropyltrimethoxysilane, an organosilane ester silane coupler	>1%	Nuclear Magnetic Resonance
Elongation	35 percent, minimum Type I specimen, thickness 0.25 ± 0.03" at Rate = 0.45 inch/minute.	ASTM D638
	Sample Conditioning: 18/25/50+5/70	ASTM D618
Tensile Strength	2,500 psi, minimum Type I specimen, thickness 0.25 ± 0.03" at Rate = 0.45 inch/minute.	ASTM D638
	2,500 psi, minimum Type I specimen, thickness 0.25 ± 0.03" at Rate = 0.45 inch/minute.	ASTM D618

6-23.2(1)C Polyester Concrete Aggregates

The polyester concrete aggregate (coarse and fine) shall be thoroughly washed and kiln dried.

Polyester concrete aggregates shall be manufactured from sand and gravel in accordance with Section 4-01. Fine aggregate shall consist of natural sand only. Reclaimed concrete aggregate shall not be used.

Polyester concrete aggregate shall have the following properties:

Polyester Concrete Aggregate Gradation	
Sieve Size	**Percent Passing**
½"	100
⅜"	98 minimum
#4	62-85
#8	45-67
#16	29-50
#30	16-36
#50	5-20
#100	0-7
#200	0-3

Properties of Polyester Concrete Aggregate		
Property	**Requirement**	**Test Method**
Los Angeles Wear	35% max at 500 rev	AASHTO T96
Degradation Factor	30 minimum	WSDOT T113
Clay lumps and Friable Particles	3.0% by weight	AASHTO M6
Coal and lignite	0.25% by weight	AASHTO M6
Particles of specific gravity less than 2.0	1.0% by weight	AASHTO M6
Crushed particles	<45% Crushed Particles, retained on the No. 8 Sieve	AASHTO T335
Weighted-average aggregate absorption	<1%	AASHTO T84 and T85
Mohs Hardness	≥7 (≥6.5 if system has demonstrated more than 10 years of success on large scale installations)	Mohs Hardness Test

Aggregate shall comply with the following properties at the time of mixing the polyester concrete:

The polyester concrete aggregate shall have a weighted-average moisture content when tested under AASHTO Test Method T255 of not more than one half of the weighted-average aggregate absorption.

6-23.2(1)D Polyester Concrete

The properties of the polyester concrete, when the polyester resin and polyester concrete aggregates are combined in the proportions of the approved mix design, shall be as follows:

Property	Requirement	Test Method
Portland Cement Concrete Saturated Surface Dry Bond Strength	500 psi minimum at 24 hrs. and 70°± 1° F (without primer, at 12% resin content by weight of the dry aggregate, on Saturated Surface Dry Specimen)	California Test 551
PCC Saturated Surface-Dry Bond Strength (Adhesive)	700 psi, minimum at 24 hours and 70° ± 1°F (at 12% resin content by weight of the dry aggregate), HMWM primed surface	California Test 551
Abrasion Resistance	<2g weight loss (at 12% resin content by weight of the dry aggregate)	California Test 550
Modulus of Elasticity	1,000,000 psi to 2,000,000psi (at 12% resin content by weight of the dry aggregate)	ASTM C 469
Portland Cement Concrete Dry Surface Bond Strength (Adhesive) – Primer installation window verification	700 psi, minimum at 24 hours and 70° ± 1°F (at 12% resin content by weight of the dry aggregate), HMWM primed surface. Polyester concrete placed against primed surface two hours after Primer application.	California Test 551

6-23.2(1)E Sand for Abrasive Finish

Sand for abrasive finish shall have the following properties:

1. Be commercial-quality blast sand.
2. Have a minimum of 85 percent passing the No. 8 sieve and a maximum of 10 percent passing the No. 20 sieve when tested under AASHTO Test Method T27.
3. Be kiln dried and protected from moisture until time of placement. At the time of application on the polyester concrete, the moisture content of the sand for abrasive finish shall not exceed 0.5 percent.

6-23.2(1)F Shipping, Storing and Handling Polyester Concrete Materials

All components shall be shipped in strong, substantial containers bearing the manufacturers label specifying batch/lot number, brand name, and quantity. If bulk resin is to be used, the contractor shall notify the Engineer in writing 10 days prior to the delivery of the bulk resin to the job site. Bulk resin is any resin that is stored in containers in excess of 250 gallons.

All materials shall be delivered in their original containers bearing the manufacturer's label, specifying date of manufacturing, batch number, trade name brand, quantity, and mixing ratio. Each shipment of polyester concrete binder and primer shall be accompanied by a Safety Data Sheet (SDS). Bulk resin containers shall be identified by one of the following methods:

1. A label on each container as specified above, or
2. A marking on each container that uniquely identifies the container, accompanied by documentation that unequivocally identifies the Manufacturer’s Certificate of Compliance that is associated with the material in that container.

The material shall be stored to prevent damage by the elements and to ensure the preservation of their quality and fitness for the Work. The storage space shall be kept clean and dry and shall contain a high-low thermometer. The temperatures of the storage space shall not fall below nor rise above that recommended by the manufacturer. Every precaution shall be taken to avoid contact with flame.

Stored materials shall be inspected prior to their use and shall meet the requirements of these Specifications at the time of use.

Material which is rejected because of failure to meet the required tests or that has been damaged shall be immediately replaced at no additional expense to the Contracting Agency.

Sufficient material to perform the entire polyester concrete overlay application shall be in storage at the site prior to field preparations, so that there shall be no delay in procuring the materials for each day's application.

Prior to Work, a copy of the Contractor’s safety plan addressing worker protective clothing, protective breathing devices, measures to address inadvertent contact with chemicals and other appropriate safety measures shall be submitted to the Engineer in accordance with Section 1-07.1(2).

6-23.2(2) Concrete Class M

Concrete Class M shall be proportioned in accordance with the following mix design:

Portland Cement Type 1 or Type 1, or Blended Hydralulic Cement Type IL(X)	705 pounds
Fine Aggregate	1,280 pounds
Coarse Aggregate	1,650 pounds
Water/Cement Ratio	0.37 maximum
Air (± 1½ percent)	6 percent
Slump (± 1 inch)	5 inches

Fine aggregate shall be Class 1. Coarse aggregate shall be AASHTO grading No. 7 or No. 8.

The use of a water-reducing admixture conforming to AASHTO M 194 Type A will be required to produce Concrete Class M with the desired slump. Air entraining admixtures shall conform to AASHTO M 154. The use of accelerating admixtures or other types of admixtures is not allowed.

Concrete Class M shall be mixed in batch-plants and transported in ready-mix trucks conforming to Section 6-02.3(4)A.

The maximum allowable and actual water/cementitious ratios shall be calculated using all the available mix water, including water added at the plant, water added in transit and at the job site, water in all admixtures, and the free water in the aggregates but not the water absorbed by the aggregates. The following are considered cementitious materials: Portland Cement and blended hydraulic cement.

6-23.2(3) Crack Sealing Materials

6-23.2(3)A Crack Sealing Resin

Resin for sealing cracks in the polyester concrete overlay shall meet the requirements for polyester concrete binder.

6-23.2(3)B Crack Sealing Sand

Sand for topping the crack sealing resin shall meet the requirements for sand for abrasive finish.

6-23.3 Construction Requirements

6-23.3(1) Sequence of Operations

The sequence of the Work shall be as follows. This sequence is in addition to other sequence and timing requirements in this Specification:

1. Shotblasting existing Bridge Deck Surface
2. Surveying of Existing Bridge Deck Surface
3. Perform Type 1 and Type 2 Deck Repair
4. Sandblast, and clean the finished surface
5. Place and cure the primer, polyester concrete overlay, and sand for abrasive finish
6. Check for bond and repair as required
7. Crack Sealing
8. Grind for smoothness
9. Texturing Polyester Concrete

6-23.3(1)A Traffic Restrictions on Sequence of Operations

Traffic shall not be allowed on shotblasted bridge deck surfaces until step 9 of Section 6-23.3(1) is completed.

6-23.3(2) Equipment

In addition to meeting the equipment requirements herein, equipment shall meet, and be operated in accordance with, the System Provider Technical Representative's recommendations.

6-23.3(2)A Shot Blaster

The shotblaster shall be a self-contained mobile unit using steel shot to texture the sound concrete to produce a concrete surface profile of CSP-6 or greater in accordance with International Concrete Repair institute (ICRI) 310.2R. The machine shall blast a minimum width of 2 feet per pass. The shotblasting machine shall shotblast, vacuum and store all material removed from the blasted concrete surface in a self-contained unit.

The shotblaster vacuum shall allow the shotblaster to be operated in air pollution sensitive areas and shall be equipped to not contaminate the deck during final preparation for concrete placement.

6-23.3(2)B Power Driven Hand Tools

Power driven hand tools are limited to the following:

1. Jack hammers no heavier than the nominal 30-pound class.
2. Chipping hammers no heavier than the nominal 15-pound class.
3. Other mechanical means acceptable to the Engineer.

Power driven hand tools shall not be operated at angles greater than 45 degrees as measured from the surface of the deck to the tool.

6-23.3(2)C Air Compressor

Air compressors shall be equipped with oil traps to eliminate oil from being blown onto the bridge deck.

6-23.3(2)D Vacuum Machine

Vacuum machines, separate from and in addition to the vacuum built in to the shot-blaster, shall be capable of collecting all remaining dust, concrete chips, and other debris encountered while vacuuming. The machines shall be equipped with collection systems that allow the machines to be operated in air pollution sensitive areas and shall be equipped to not contaminate the deck during final preparation for concrete placement.

6-23.3(2)E Polyester Concrete Mixers

A continuous automated mixer shall be used for all polyester concrete overlay applications. The continuous mixer must be capable of mixing the polyester binder resin components with dry aggregate, maintain proper ratios, and achieve set and cure times within the specified limits.

The Contractor shall submit current certification documents showing that mixing equipment has been calibrated (California Test 109 or similar accepted) with the exact polyester concrete overlay system to be installed. If required by the Engineer, the Contractor shall demonstrate that the proposed volumetric mixing equipment is accurately calibrated through on-site verification. The actual weights of the polyester concrete materials discharged from the volumetric mixer truck shall be accurately represented by the printout ticket measurement produced by the on-board computer tracking system. To demonstrate this the Contractor shall dispense individual aggregate and resin batches and weigh with certified scales. The Engineer will compare certified scale weights to print out ticket measurements. Results of each comparison will be considered within calibration tolerance when ticket measurements and certified scale weights are within 2% of each other. Mixing equipment calibration verification should be considered successfully completed after three consecutive successful results, witnessed by a representative of the Contracting Agency.

The Contractor shall submit a documented history of the use of the placement equipment to successfully install Polyester Polymer Concrete overlays on bridge projects for review and approval by the Engineer. Acceptable experience shall be from installations matching the scope of the proposed project, including thickness and grade establishment requirements.

The continuous mixer shall:

1. Employ an auger screw/chute device capable of sufficiently mixing catalyzed resin with dry aggregate.
2. Employ a plural component pumping system capable of handling polyester binder resin and additives while maintaining proper ratios to achieve set/cure times within the specified limits, evenly across the placement. Resin and all field additives, including catalyst and accelerator, shall flow through a static mix tube for sufficient duration to completely mix the liquid system prior to combination with aggregates.
3. Be equipped with an automatic metering device that measures and records aggregate and resin volumes. Record volumes at least every five minutes, including time and date. Submit recorded volumes at the end of the work shift.
4. Have a visible readout gage that displays running totals of aggregate and resin being recorded.
5. Produce a satisfactory mix consistently during the entire placement, and maintain appropriate resin content, catalyst, and accelerator levels to produce desired outcome.
6. Discharge mixed material directly into the finishing machine.

A portable mechanical mixer of appropriate size for proposed batches, as recommended by the System Provider Technical Representative and approved by the Engineer, may be used for patching applications and for smaller area overlay applications if recommended by the System Provider Technical Representative and approved by the Engineer.

6-23.3(2)F Polyester Concrete Paving Machine

Except under the conditions described in Section 6-23.3(2)F1, the polyester concrete overlay shall be placed with a self-propelled slip-form paving machine that places, consolidates, and finishes the polyester concrete overlay in one continuous operation. It shall be modified or specifically built to effectively place the polyester concrete overlay in a manner that meets Contract requirements. In addition, the paving machine shall:

1. Employ a vibrating pan to consolidate and finish the polyester concrete. Paver primary finishing pan size shall measure not less than 2 feet in the dimension parallel to the direction of paver travel. Secondary profile finishing attachments, bolt on sections, and trailing pan extensions shall not be included in this measurement.
2. Shall have the necessary adjustments to produce the required cross section, line, and grade, including the ability to recreate transverse grade breaks within 6 inches left or right of existing transverse grade breaks.
3. Be fitted with hydraulically controlled grade automation devices on both sides of the machine to establish the finished profile and cross-slope. These devices shall either (1) average 15 feet in front and behind the center of automation sensors, or (2) the sensor shall be constructed to work with string-line control. It is acceptable to match grade when placing lanes adjacent to polyester concrete overlay placed on

this Contract. String line grade establishment may be required to establish proposed grades if required by plan note or elsewhere in the Contract, in which case grade averaging beams will not be acceptable.

4. Have sufficient engine power and weight to provide adequate vibration of the finishing pan while maintaining consistent forward placement speed.
5. Be capable of both forward and reverse motion under its own power.
6. Demonstrate successful performance with the trial overlay.

Wheel or rubber tire mounted paving machines will not be allowed.

6-23.3(2)F1 Vibratory Screed and Small Surfaces

Roller type screeds will not be accepted.

A vibratory screed riding on preset forms or rails set at a maximum width of 12 feet may be used on structures that have live load paving train restrictions.

Shoulder pours of 6 feet wide or less may be placed without the use of a paving machine.

Finishing of patch areas shall be completed using hand concrete finishing tools. Patches shall be placed flush with the top of the existing deck surface.

6-23.3(2)G Smoothness Grinding Equipment

Equipment for grinding polyester concrete overlay that does not meet the surface smoothness requirements shall use diamond embedded saw blades gang mounted on a self-propelled machine that is specifically designed to smooth and texture concrete pavement or polyester concrete overlays. The equipment shall not damage the underlying surface, cause fracture, or spalling of any joints. The final surface texture shall be uniform in appearance with longitudinal corduroy type texture. The grooves shall be between 3/32 and 5/32 inches wide, and no deeper than 1/16 inch. The land area between the grooves shall be between 1/16 and ⅛ inches wide.

6-23.3(2)H Texturing Equipment

Equipment for texturing the polyester concrete overlay shall use diamond tipped saw blades mounted on a power driven, self-propelled machine that is designed to texture concrete surfaces. The grooving equipment shall provide grooves that are ⅛" ± 1/64" wide, 3/16" ± 1/16" deep, and spaced at ¾" ± ⅛".

In locations where saw cutting cannot be done the Contractor is allowed to use the spring tining method for texturing. The spring tining shall provide the same groove, spacing and depth of the saw cut texture.

The Contractor shall demonstrate that the method and equipment for texturing the bridge deck will not chip, spall or otherwise damage the overlay.

6-23.3(3) Submittals

The Contractor shall submit the following Working Drawings in accordance with Section 1-05.3:

1. A Type 2 Working Drawing of the shot-blasting equipment with associated background information and catalog cuts.
2. A Type 2 Working Drawing of the Debris Containment and Disposal Plan. This plan shall describe the methods and materials used to contain, collect, and dispose of all concrete debris generated by all operations, including but not limited to shotblasting, Type 1 Deck Repair, Type 2 Deck Repair, sandblasting, and cleaning. The Working Drawing shall also address provisions for protecting adjacent traffic from flying debris.

3. A Type 2 Working Drawing of the polyester concrete mix design meeting the requirements of Section 6-23.2(1). The mix design shall include a recommended initiator percentage for the expected application temperature.
4. A Type 1 Working Drawing of the mix design for concrete Class M. This submittal shall be on WSDOT Form 350-040 and shall provide a unique identification for each mix design. A unique identification for the mix design is composed of the combination of the Mix Design Number and the Concrete Plant Number.
5. A Type 2 Working Drawing of samples, as specified below, shall be submitted to the Engineer at least 15 working days prior to placing the polyester overlay:
    a. One gallon minimum of the polyester concrete binder.
    b. One pint minimum of the primer.
    c. 100 pounds minimum of polyester concrete aggregate.
6. A Type 2 Working Drawing of the paving equipment specifications and details of how the paver will maintain the required longitudinal and transverse grades.
7. A Type 1 Working Drawing of the survey data collected as required in Section 6-23.3(6).
8. A Type 1 Working Drawing of the measurements documenting the deck patching areas as required by Section 6-23.3(7)B.
9. A one-pint sample of each batch of promoted/initiated primer shall be retained and submitted to the Engineer at the time of primer application to verify proper catalyzation.
10. A Type 1 Working Drawing of the readings of the rebound hammer used shall be correlated to the compressive strength of the polyester concrete product in accordance with Section 5.4 of ASTM C805 and the Contractor.
11. A Type 2 Working Drawing of the qualifications of on-site supervisors, volumetric mixer operators, and finishing machine operators, in accordance with Section 6-23.1(2)C.
12. A Type 2 Working Drawing of the method and materials used to contain primer and polyester concrete within the deck area specified to receive the overlay.
13. A Type 2 Working Drawing of the Contractor's Safety plan addressing worker protective clothing, protective breathing devices, measures to address inadvertent contact with chemicals and other appropriate safety measures.
14. A Type 2 Working Drawing of the equipment to be used for texturing.
15. A Type 2 Working Drawing of the Certified test results as required in Section 6-23.2(1).
16. A Type 1 Working Drawing of the Documentation of the System Provider Technical Representative's experience, demonstrating compliance with the experience requirements, including the following:
    a. Years of Experience with the proposed Polyester Concrete Overlay System
    b. Project location
    c. Project construction date
    d. Overlay quantities
    e. Reference name and contact information for owner representative

17. A Type 2 Working Drawing of the Documentation of the Polyester Concrete Overlay System and System Provider experience, demonstrating compliance with experience requirements. Submit written installation instructions, safety data sheets, and independent test results for approval. Projects of similar scope shall be evaluated considering placement temperature, traffic return, allowable cure time, placement thickness, average daily traffic, surface texture, environmental conditions, and any other factors unique to the application. System failure examples obtained from other Public Agencies may be considered for evaluation and rejection whether submitted by the Contractor or obtained otherwise. Submit documentation and references of the polyester concrete overlay system experience including the following:
    a. Project location
    b. Contracting Agency
    c. Project construction date
    d. Overlay quantities and component details
    e. Reference name and contact information for owner representative
18. A Type 2 Working Drawing of the Documentation of the experience of the Polyester Concrete Placement Contractor and Workers that will place the polyester concrete overlay system. The documentation of Contractor and employee qualifications shall include the following:
    a. Project location
    b. Contracting Agency
    c. Project construction date
    d. Overlay volume and area quantities
    e. Reference name and contact information for owner representative
19. A Type 2 Working Drawing of the certification and test reports of the polyester concrete mixer and documented history of the use of the placement equipment to successfully install Polyester Polymer Concrete overlays.
20. A Type 2 Working Drawing of the Overlay Placement Plan. The Contractor shall submit an Overlay Placement Plan that includes the following:
    a. Schedule of overlay work and testing for each bridge
    b. Staging plan describing overlay placement sequence including:
        i. Construction joint locations
        ii. Sequence of placement
        iii. Paving widths
        iv. Anticipated paving lengths
        v. Paving directions
        vi. Joint locations
        vii. Location of proposed trial overlay(s)
    c. Description of equipment used for:
        i. Surface preparation including grinding and shot blasting
        ii. Applying primer
        iii. Measuring, mixing, placing, and finishing the polyester concrete overlay
        iv. Applying sand for abrasive finish

   d. Method of protecting and finishing inlets and bridge drains
   e. Method for isolating expansion joints
   f. Method for ensuring shotblasting achieves a concrete surface profile of ICRI 310.2R CSP-6 or greater
   g. Method for measuring and maintaining overlay thickness and profile
   h. Cure time for polyester concrete
   i. Storage and handling of primer and polyester concrete components
   j. Procedure for disposal of excess primer, polyester concrete, and containers
   k. Procedure for cleanup of mixing and placement equipment

6-23.3(4) Operations on the Bridge Deck

The following apply to all Contractor operations on the bridge deck, including but not limited to cleaning concrete surfaces, Type 1 and Type 2 Deck Repair, sandblasting, shot-blasting, placing, consolidating, finishing, curing, sawing, and crack sealing the overlay.

1. The Contractor shall not use water on the bridge deck nor allow water from their operations to come into contact with the concrete bridge deck at any time, except for the following:
   a. Placing and curing Class M concrete. Using water for this application shall be carefully controlled to prevent the water from coming into contact with the bridge deck outside of the patch.
2. The Contractor shall protect adjacent traffic from flying debris in accordance with its Debris Containment and Disposal Plan submitted in accordance with Section 6-23.3(3).
3. The Contractor shall collect, contain, and dispose of all concrete debris in accordance with its Debris Containment and Disposal Plan submitted in accordance with Section 6-23.3(3).
4. Rainwater and stormwater runoff that comes in contact with the bridge deck shall be considered process wastewater and shall be managed in accordance with Section 8-01.

6-23.3(5) Initial Surface Preparation

Initial surface preparation is for the purpose of exposing the concrete substrate for chain dragging and deck repair.

6-23.3(5)A Prerequisites to Initial Surface Preparation

Initial surface preparation shall not begin until the Contractor has completed all the following:

1. Demonstrated that all Work, for a given bridge, needed to complete items 1, 2, 3, 4, 5, 6, 7, 8, and 9 of Section 6-23.3(1) can and will be completed in one and only one construction season.
2. Submitted all submittals required in Section 6-23.3(3) and addressed all the Engineer's comments to the satisfaction of the Engineer.

6-23.3(5)B Shotblasting

For newly constructed bridge decks, the deck concrete shall cure a minimum of 28 days and attain design concrete compressive strength prior to shotblasting.

The areas to receive polyester concrete overlay shall be shotblasted, or sandblasted if the shotblast equipment cannot access areas to be prepared, to produce a concrete surface profile of CSP-6 or greater in accordance with International Concrete Repair Institute

(ICRI) 310.2R. All weak or loose surface mortar shall be removed, aggregates within the concrete exposed, and open pores in the concrete exposed, as well as a visible change in the concrete color.

Dust and debris generated during shotblasting shall be picked up and stored in the vacuum unit built into the shotblaster and minimal dust shall be created during the blasting operation.

6-23.3(6) Surveying of Existing Bridge Deck

After shotblasting the concrete surface as specified in these Specifications, the Contractor shall complete a survey of the Existing Bridge Deck Surface(s) specified to receive Polyester concrete overlay for use in establishing the existing cross section and profile grade elevations.

The Engineer will provide the Contractor with primary survey control information consisting of descriptions of two primary control points used for the horizontal and vertical control. Primary control points will be described by reference to the bridge or project-specific stationing and elevation datum. The Engineer will also provide horizontal coordinates for the beginning and ending points and for each Point of Intersection (PI) on each centerline alignment included in the project. The Contractor shall provide the Engineer 21 calendar days' notice in advance of scheduled concrete surface shotblasting work to allow the Engineer time to provide the primary survey control information.

The Contractor shall verify the primary survey control information furnished by the Engineer and shall expand the survey control information to include secondary horizontal and vertical control points as needed for the project. The Contractor's survey records shall include descriptions of all survey control points, including coordinates and elevations of all secondary control points.

The Contractor shall maintain detailed survey records, including a description of the work performed on each shift, the methods utilized to conduct the survey, and the control points used. The record shall be of sufficient detail to allow the survey to be reproduced. A Type 1 Working Drawing of each day's survey record shall be provided to the Engineer within 3 working days after the end of the shift. The Contractor shall compile the survey information in an electronic file format acceptable to the Engineer (file formats submitted shall be compatible with InRoads and MicroStation).

Survey information collected shall include station, offset, and elevation for each lane line and curb line. Survey information shall be collected at even 20-foot station intervals and at the centerline of each bridge expansion joint. The Contractor shall ensure a surveying accuracy to within ± 0.01 feet for vertical control and ± 0.2 feet for horizontal control. The survey shall extend 100 feet beyond the bridge back of pavement seat.

Except for the primary survey control information and final grade profile and cross-section furnished by the Engineer, the Contractor shall be responsible for all calculations, surveying, and measuring required for setting, maintaining, and resetting equipment and materials necessary for the construction of the overlay to the final grade profile and cross-section. The Engineer may post-check the Contractor's surveying, but these post-checks shall not relieve the Contractor of responsibility for internal survey quality control.

The Engineer will establish the final grade profile and cross-section based on the Contractor's survey and will provide the final grade profile and cross-section to the Contractor within five working days after receiving the Contractor's survey information.

The Contractor shall not begin shotblasting concrete surface work as specified in these Specifications until receiving the final grade profile from the Engineer.

6-23.3(7) Deck Repair

Deck repair Work shall not commence until shotblasting operations are complete.

6-23.3(7)A Classification

Deck repair will be classified as Type 1 Deck Repair or Type 2 Deck Repair. The determination of whether an area will be classified as Type 1 or Type 2 will be made after completion of deck repair excavation, repair of steel reinforcing bars, and removal of concrete debris.

6-23.3(7)B Chain Drag

After the entire lane or strip to be overlaid has been shotblasted and cleaned as required in Section 6-23.3(5), the entire surface shall be inspected by the Contractor, in the presence of the Engineer, in accordance with ASTM D4580, Method B. Based on that inspection, the Contractor shall mark those areas, meeting any of the following criteria, for removal:

1. Unsound concrete in accordance with ASTM D4580, Method B.
2. Lack of bond between existing concrete and reinforcing steel.
3. All existing nonconcrete patches.

After all deck repair excavation is complete, the Contractor shall measure and submit to the Engineer as a Type 1 Working Drawing the location and size of each area identified above by station, offset, length, width, average depth, and deck repair type, using the form provided by the Engineer.

6-23.3(7)C Deck Repair Excavation

The areas marked for removal in Section 6-23.3(7)B shall be excavated with equipment as described in Section 6-23.3(2)B. Excavation shall be to the depth necessary to remove all loose and unsound material, without damaging reinforcing steel or sound concrete.

Care shall be taken in removing the deteriorated material to not damage the existing sound concrete or steel reinforcing bars that are to remain in place. All removal shall be accomplished by making vertical edges at the boundaries of the repair area. In no case shall the depth of a sawn vertical cut exceed ¾ inch or to the top of the top steel reinforcing bars, whichever is less.

Bridge deck areas outside the repair area damaged by the Contractor’s operations shall be repaired by the Contractor at no additional expense to the Contracting Agency, and to the satisfaction of the Engineer.

6-23.3(7)D Repair of Steel Reinforcing Bars

Where existing steel reinforcing bars inside deck repair areas show natural deterioration greater than 20-percent section loss, the Contractor shall furnish and place steel reinforcing bars alongside the deteriorated bars in accordance with the details shown in the Standard Plans. Payment for such extra Work will be by force account as provided in Section 1-09.6.

All reinforcing steel damaged due to the Contractor’s operations shall be repaired by the Contractor. Damage to rebar shall be understood to include damage to epoxy coating.

The repair shall be as follows or as directed by the Engineer:

1. Damage to epoxy coating, when present on existing steel reinforcing bars, shall be repaired in accordance with Section 6-02.3(24)H.
2. Damage to bars resulting in a section loss of 20 percent or more of the bar area shall be repaired by chipping out the adjacent concrete and splicing a new bar of the same size. Concrete shall be removed to provide a ¾-inch minimum clearance around the bars. The splice bars shall extend a minimum of 40 bar diameters beyond each end of the damage.
3. All bars partially or completely removed from the deck shall have the damaged portions removed and spliced with new bars as outlined in item 2 above.

For bridge decks not constructed under the same Contract as the polyester concrete overlay, responsibility for costs to repair damage shall be allocated as follows:

1. Repairing damage that occurs during shotblasting to coatings on existing reinforcing steel shall be paid for in accordance with Section 1-09.6.
2. Repairing damage to existing reinforcing steel that is caused by the Contractor's negligence shall be at no additional expense to the Contracting Agency.

6-23.3(7)E Type 1 Deck Repair

An area will be classified as a Type 1 Deck Repair when the completed concrete excavation either (a) exposes no more than one-half the periphery of a bottom bar of the top layer of steel reinforcement, or (b) the length of an exposed bar does not exceed 12-continuous inches along the length of the bar.

The scope of Work for Type 1 Deck Repair includes:

1. Excavating and disposing of the unsound concrete and unsound nonconcrete patches within the repair area.
2. Repair of steel reinforcing bars damaged by the Contractor.
3. Sandblast the surface and exposed rebar.
4. Providing a CSP-6 surface roughness on existing nonconcrete patches that are sound.

6-23.3(7)F Type 2 Deck Repair

An area will be classified as a Type 2 Deck Repair when the completed concrete excavation either exposes more than one-half the periphery of a bottom bar of the top layer of steel reinforcement or the length of an exposed bar exceeds 12-continuous inches along the length of the bar.

The scope of Work for Type 2 Deck Repair includes:

1. Excavating and disposing of the unsound concrete and unsound nonconcrete patches within the repair area, below the shotblasted depth.
2. Repairing steel reinforcing bars damaged by the Contractor.
3. Sandblasting the area and exposed rebar prior to placing deck patching concrete.
4. Saturating and removing freestanding water.
5. All work related to patching and curing the excavated area with Class M concrete in accordance with Section 6-23.2(2).

6-23.3(7)G Filling and Curing Deck Repair Areas

Type 1 Deck Repairs shall be filled with polyester concrete as part of placing the polyester concrete overlay. Payment for filling Type 1 deck repairs with Polyester Concrete shall be incidental to bid item "Polyester Concrete Overlay".

Type 2 Deck Repairs shall be patched with concrete class M. The top of these patches shall be finished with a wood float, flush with the top of the shotblasted surface. All Type 2 deck repair patching shall be performed well enough in advance of the polyester concrete overlay to allow all patches to cure as required below.

Before placing Class M concrete in the Type 2 deck repairs, the Contractor shall clean the surfaces to which the concrete will be applied (including rebar) by sandblasting and blowing clean with oil-free air. The Contractor shall make sure the existing concrete is well saturated at the time of placing concrete in the Type 2 deck repairs but shall remove all freestanding water prior to placing the concrete. The Contractor shall place concrete class M in the Type 2 deck repair areas while the existing concrete is wet. It shall be consolidated in accordance with Section 6-02.3(8). Concrete Class M shall be wet-cured a minimum of 42 hours, as follows:

1. The concrete shall be immediately covered with a single layer of clean, new or used, wet burlap. The burlap shall have a maximum width of 6 feet. The Engineer will determine the suitability of the burlap for reuse, based on the cleanliness and absorption ability of the burlap. Care shall be exercised to ensure that the burlap is well drained and laid flat with no wrinkles on the deck surface. Adjacent strips of burlap shall have a minimum overlap of 6 inches.
2. Once in place the burlap shall be lightly fog sprayed with water. A separate layer of white, reflective type polyethylene sheeting shall immediately be placed over the wet burlap.
3. As an alternative to the application of burlap and fog spraying described above, the Contractor may propose a curing system using proprietary curing blankets specifically manufactured for bridge deck curing. The Contractor shall submit a Type 2 Working Drawing consisting of details of the proprietary curing blanket system, including product literature and details of how the system is to be installed and maintained.
4. The burlap shall be kept wet continuously and the wet curing regimen as described shall remain in place for a minimum of 42-hours.

During the curing period of concrete placed in Type 2 deck repairs, all vehicular and foot traffic shall be prohibited in the repair area.

6-23.3(7)H Filling Existing Bridge Deck Wheel Ruts

Existing Bridge Deck Ruts shall be filled with polyester concrete as part of placing the polyester concrete overlay.

6-23.3(8) Polyester Concrete Trial Overlay

Prior to constructing the overlay, the Contractor shall place one or more trial overlays of primer and polyester concrete using the equipment, materials, and procedures proposed for production, as approved by the Engineer in accordance with Section 6-23.3(3). The Contractor shall notify the Engineer of the time and location of the trial overlay at least seven calendar days prior to the scheduled trial overlay.

The trial overlay shall be placed on a previously cast and cured concrete pad at a location selected by the Contractor. The plan area of the concrete pad shall be 12 feet minimum in width and 15 feet minimum in length.

The Contractor shall shotblast, clean the concrete pad surface, mix, place, finish, and cure the polyester concrete overlay. The Contractor need not perform further deck preparation, or place sand for abrasive finish provided that all other conditions of Sections 6-23.3(9), (10), and (12) are satisfied.

The Contractor shall arrange for soundness testing and three pull-off tests as described in Section 6-23.3(13) to be performed by an independent testing laboratory. The independent testing laboratory shall record the pull-off test results and the amount of (if any) failure into the base concrete and shall provide written documentation of the test results to the Engineer and Contractor.

The Contractor shall not begin placing polyester concrete overlay at the bridge site(s) receiving the polyester concrete overlay until receiving the Engineer's approval of the completed trial overlay.

After receiving the Engineer's approval of the completed trial overlay, the concrete pad and trial overlay shall become the Contractor's property and shall be removed and disposed of in accordance with Section 3-02.3.

If significant successful experience is demonstrated by both the installer, System Provider, and System Provider Technical Representative together, the first shift of polyester concrete overlay installation may be considered as the Trial Application if approved by the Engineer. Rejection of all or part of the trial in this case will be required to be removed and disposed of at no additional cost to the Contracting Agency. If no further overlay is allowed due to full rejection after multiple trials, the site will be restored to initial in-service condition at no additional cost to the Contracting Agency.

The number of trial applications required shall be as many as necessary for the Contractor to demonstrate the ability to construct an acceptable trial overlay section and competency to perform the work. However, the installer, proposed equipment/techniques, or material may be rejected if not shown to be acceptable after two trials.

6-23.3(9) Polyester Concrete Overlay

6-23.3(9)A Pre-Overlay Conference

Five to ten working days prior to polyester concrete overlay placement, a pre-overlay conference shall be held to discuss final deck preparation, equipment, temperature and weather requirements, aggregate and deck dryness requirements, construction procedures, sequencing, and personnel. Inspection procedures shall also be reviewed to ensure coordination. Attendees shall include representatives from all parties involved in the work including inspectors, installer, and System Provider Technical Representative. If necessary, teleconferencing of attendees may be approved by the Engineer.

If the project includes more than one bridge deck, an additional conference shall be held just before placing the polyester concrete overlay for each subsequent bridge deck.

6-23.3(9)B Restrictions on Other Work

To ensure the best possible bond and integrity of the polyester concrete overlay, the Contractor shall ensure that dust, debris, moisture, or any other deleterious materials do not enter a work area from the start of final surface preparation in that work area until completion of curing time for the polyester concrete overlay in that work area. This work area during this timeframe shall be referred to as the protected work area. In addition to other measures, the Contractor shall comply with the following:

1. Perform no work within 100 feet of the protected work area which generates dust or debris (including hand tool chipping, shotblasting, sandblasting, vacuuming, and cleaning).
2. Dust or debris generating work may be allowed beyond this 100 feet boundary provided dust and debris will not drift onto the limits of the protected work area.

If the shotblasting impedes or interferes in any way with the final cleaning or overlay placement within the protected area as determined by the Engineer, the shotblasting Work shall be terminated immediately and the equipment shall be moved away from the protected area to eliminate the conflict.

Traffic other than required construction equipment will not be permitted within the protected work area unless allowed by the Engineer. To prevent contamination, all equipment allowed within the protected work area shall be equipped with drip guards.

6-23.3(9)C Final Surface Preparation

Following the completion of all Type 1 and Type 2 Deck Repairs (including placement and curing of patches in Type 2 Deck Repair areas), the entire lane or strip being overlaid shall undergo final cleaning. Final cleaning shall be accomplished in one shift and consists of the following, in the sequence shown:

1. Remove grease, slurry, oils, paint, dirt, striping, cure compound, rust, membrane, milling slurry, weak surface mortar or any other contaminants that could interfere with the proper adhesion of the overlay system. These materials shall be removed by abrasive blasting.
2. All steel surfaces that will be in contact with the overlay shall be cleaned in accordance with SSPC-SP No. 10, Near-White Blast Cleaning, except that wet blasting methods shall not be allowed.

3. Remove loose or trapped particles using magnets and vacuuming. Vacuum shall be capable of collecting all remaining dust, concrete chips, and other debris to the extent necessary to ensure the oil-free compressed air in the next step complies with environmental requirements.
4. Oil-free compressed air shall be used as the final step to remove all remaining dust and debris.
5. Cleaned surfaces shall not be exposed to Contractor or public vehicular traffic. If the deck becomes contaminated before placing the overlay, the Contractor shall shotblast or sandblast the contaminated areas to the satisfaction of the Engineer at no additional cost to the Contracting Agency.
6. The Contractor shall provide suitable coverings (e.g. heavy duty drop cloths) as needed to protect all exposed areas not to receive primer and overlay, such as curbs, sidewalks, parapets, etc. All damage or defacement resulting from this application shall be cleaned and/or repaired to the Engineer's satisfaction at no additional cost.

6-23.3(9)D Overlay Finishing Equipment Setup

Construction joints between passes shall be within 1 foot of the stripe lines or centered within a lane.

When grade will be established for a paving machine from a paving wire, or when a vibrating screed is allowed, grade pins and screed rails shall be placed outside the area to be overlaid. Hold-down devices shot into the concrete are not permitted. Hold-down devices of other types leaving holes in the exposed area will be allowed provided the holes are subsequently filled with polyester concrete. Hold-down devices shall not penetrate the existing deck by more than ¾ inch.

6-23.3(9)E Quality Assurance for Polyester Concrete Overlay

All acceptance testing shall be performed by an independent testing laboratory provided by the Contractor, in the presence of the Engineer's representative. The Engineer reserves the right to self-perform any acceptance tests it deems in its best interests. The Contractor's independent testing laboratory shall perform the following tests:

1. Moisture content of polyester concrete aggregate and sand for abrasive finish.
2. Temperature of deck surface and aggregates before mixing.
3. ASTM C805 Rebound Hammer (Schmidt hammer).
4. Smoothness quality testing.
5. Sounding using ASTM D4580, Method B.
6. Direct Tension Bond Testing, ASTM C1583.

The Contractor shall arrange to have the System Provider Technical Representative furnish technical service relating to application of material and health and safety training for personnel who are to handle the polyester concrete and the primer, at the following times:

1. At the pre-paving conference.
2. During the trial overlay.
3. During paving machine setup.
4. During a minimum of the first two days of paving.

6-23.3(9)F Moisture and Temperature Requirements

It is critically important for the long-term performance of the polyester concrete system that the concrete substrate and all other surfaces (primer and polyester overlay) be (1) at the proper temperature and (2) moisture-free. Unless otherwise noted below, the time period for these requirements begins with the start of applying primer and ends two hours after placing the polyester overlay and sand for abrasive finish. Therefore, the following requirements for temperature and moisture shall be strictly enforced. Failure to follow these requirements may result in removal and replacement of the polyester concrete system at no additional expense to the Contracting Agency.

1. During the 24-hour period immediately preceding start of primer placement, the area of bridge deck to receive primer shall not be exposed to moisture or water in any form. Additionally, during this 24-hour period, the concrete substrate shall be exposed to the atmosphere to freely allow moisture to evaporate. Covering the concrete substrate during this period with material that will hinder evaporation in any way, such as visqueen, shall not be allowed.
2. Primer application shall not begin if rain is forecast any time between start of primer application and 2 hours after the planned completion of polyester concrete and sand for abrasive surface.
3. Primer application shall not begin until after morning dew has evaporated.
4. Before starting primer, the concrete substrate surface must be free of any surface darkening that would indicate locations of previously standing water. The entire concrete substrate surface must appear to be uniformly light in color and show no further lightening when drying methods such as blowing compressed air are applied. Cracks in the concrete substrate must also be dry.
5. The concrete surface temperature shall be between 40°F (and rising) and 100°F. Night work may be required when temperatures cannot be met during the day.

6-23.3(9)G Primer Application

The primer placement shall start not more than 24 hours after the start of sandblasting operations in Final Surface Preparation.

In the interim between completion of final surface preparation described in Section 6-23.3(9)C and applying the primer, any contaminants that have accumulated which could interfere with the proper adhesion of the overlay system shall be removed to the satisfaction of the Engineer. Immediately prior to applying the primer, the surface receiving the primer shall be blown off with oil free and moisture free compressed air to remove accumulated dust and any other loose material.

After the exposed surfaces have been prepared and are dry, primer shall be applied in accordance with the System Provider Technical Representative's recommendations. Primer shall be placed within 5 minutes of mixing at approximately 90 sf/gal or the rate that provides substrate saturation acceptable to the Engineer.

Primer shall be applied by flooding and uniformly spread to completely cover surfaces to receive overlay. Care shall be taken to avoid heavy application that results in excess puddling. Excess material shall be removed or distributed to meet the required saturation without excessive puddling. Primer shall be reapplied to any areas that appear dry 15 minutes after primer placement, prior to overlay placement.

The prepared concrete surface shall receive one coat of promoted/initiated primer. The promoted/initiated primer shall be worked into the concrete in a manner to effect complete coverage of the area. A one-pint sample of each batch of promoted/initiated primer shall be retained and submitted to the Engineer at the time of primer application to verify proper catalyzation.

Under no circumstances shall resin be allowed to run into drains and expansion joints, or otherwise escape the Contractor’s collection and containment system.

If the primed surface becomes contaminated, the contaminated area shall be cleaned by abrasive blasting and reprimed at no additional expense to the Contracting Agency. The primer shall cure for a minimum of 30 minutes before placing the polyester concrete overlay.

6-23.3(9)H Mixing Polyester Concrete

Polyester concrete shall be mixed in volumetric mixers conforming to Section 6-23.3(2)E and in accordance with the mix design accepted by the Engineer.

At the time of mixing, the polyester concrete aggregate shall:

1. Have a temperature between 45°F and 100°F.
2. Have a weighted-average moisture content, when tested under AASHTO Test Method T255, of not more than one half of the weighted-average aggregate absorption.

The amount of peroxide initiator used shall result in a polyester concrete set time between 30- and 120-minutes during placement as determined by California Test 551, Part 2, "Method of Test For Determination of Set Time of Concrete Overlay and Patching Materials", by Gilmore Needles. Accelerators or inhibitors may be required as recommended by the polyester concrete binder supplier.

The polyester concrete binder shall be initiated and thoroughly blended just prior to mixing the polyester concrete aggregate and binder. The polyester concrete shall be thoroughly mixed prior to placing.

6-23.3(9)I Placing Polyester Concrete

The polyester concrete overlay shall be placed, consolidated, and finished to the profile grade and cross-section provided by the Engineer using a paving machine meeting the requirements of Section 6-23.3(2)F. The Contractor shall perform a dry run with the paving machine before placing Polyester Concrete. Based on the dry run, adjustments to the final grade may be allowed provided minimum thickness requirements are met.

The minimum thickness of polyester concrete overlay system shall be ¾ inches, measured from the top of the Polyester Overlay to the highest point of the shotblasted concrete surface as shown in the Plans.

Placement of the polyester concrete shall not proceed until the Engineer verifies that the primer was properly promoted and initiated, as evidenced by the primer batch sample.

During overlay application, the Contractor shall provide suitable coverings (e.g., heavy duty drop cloths) as needed to protect all exposed areas not to receive overlay, such as curbs, sidewalks, parapets, etc. All damage or defacement resulting from this application shall be cleaned and/or repaired to the Engineer's satisfaction at no additional cost.

The polyester concrete shall be placed on the primer after 15 minutes and within 2 hours after the primer has been applied. The polyester concrete shall be placed prior to gelling or 15 minutes following addition of initiator, whichever occurs first.

Polyester concrete shall have an initial set time of at least 20 minutes and at most 90 minutes following resin catalyzation. The initial set time can be determined in the field when the in-place polyester concrete cannot be deformed by pressing with a finger, indicating that the resin binder is no longer in a liquid state. If the initial set is not within 90 minutes of catalyzation, the material shall be removed and replaced at no additional cost to the Contracting Agency.

If, for any reason, polyester concrete is not placed over the primer within the two-hour time limit, the Contractor shall apply a fresh coat of primer. Prior to applying the polyester concrete overlay, the surface shall be re-cleaned in accordance with Section 6-23.3(9)G.

Expansion joints shall be protected from all polyester concrete overlay operations to the satisfaction the Engineer. Saw cutting at bridge expansion joints shall not be allowed. The surface temperature of the area receiving the polyester concrete shall be the same as specified for the primer.

6-23.3(10) Finishing Polyester Concrete

The finished surface of the polyester concrete overlay shall conform to the straight-edge requirements of Section 6-23.3(15) and the following:

1. The polyester concrete shall be struck off, finished, and consolidated in accordance with the profile grade and cross-section provided by the Engineer with adjustments allowed in Section 6-23.3(9)I.
2. Binder content shall be as specified in Section 6-23.2(1)B and yield a polyester concrete consistency that requires surface applied consolidation and finishing to consolidate the polyester concrete and yield a slight sheen of bleed binder on top surface yet does not yield excess bleed binder.
3. Although the paver should yield a finished surface, additional finishing may be necessary. Hand finishing of seam area between passes shall produce a consistent surface across the junction of the placements. Polyester concrete shall be finished as necessary through traditional concrete finishing methods, producing a smooth surface, with slight resin sheen indicating complete consolidation of aggregates. Polyester concrete patches shall be finished by traditional concrete hand finishing methods.

6-23.3(11) Sand for Abrasive Finish

The polyester concrete overlay shall receive an abrasive finish using sand as specified. The abrasive finish shall be applied immediately after overlay strike-off and before gelling occurs. Where spring tining is allowed, the tining shall be performed after sufficient sand broadcast.

At the time of application on the polyester concrete, the moisture content of the sand for abrasive finish shall not exceed 0.5 percent.

At least 2.2 lbs. per square yard shall be applied evenly to refusal by hand broadcasting onto the glossy surface immediately after sufficient finishing and before resin gelling occurs. To ensure adequate pavement friction, the completed polyester concrete overlay surface (including the sand for abrasive finish) shall be free of any smooth or "glassy" areas such as those resulting from insufficient quantities of surface aggregate. Any such surface defects shall be repaired by the Contractor in the manner recommended by the System Provider Technical Representative and approved by the Engineer at no additional cost to the Contracting Agency.

6-23.3(12) Curing Polyester Concrete

The polyester concrete overlay shall be cured in accordance with the manufacturer’s recommendations. Protect the overlay from moisture, traffic, and equipment for at least 2 hours after final finishing. The Engineer may extend protection time if sufficient strength or adhesion is not achieved. The in-place material must achieve test reading from a calibrated Schmidt Hammer of at least 3,000 psi within four hours after final finishing, and before traffic or equipment is allowed on the overlay. Proper cure rate necessary to achieve sufficient initial and final strength depends on proper initiator/accelerator levels to account for field conditions such as ambient and substrate temperatures.

The Contractor shall measure the compressive strength of the cured polyester concrete overlay with a rebound hammer in accordance with ASTM C805. The readings of the rebound hammer used shall be correlated to the compressive strength of the polyester concrete product in accordance with ASTM C805 Section 5.4 and the Contractor shall submit a Type 1 Working Drawing of this correlation.

Traffic and equipment shall not be permitted on the polyester concrete overlay for at least four hours and until the polyester overlay has reached a minimum compressive strength of 3,000 psi based on the rebound hammer readings and the correlation chart for the rebound hammer used.

Areas in the polyester concrete that do not totally cure, or that fail to attain the minimum compressive strength specified above, shall have the deficiencies addressed in accordance with Section 1-05.7.

The Contractor shall prevent any cleaning chemicals from reaching the polyester mix during the overlay applications.

6-23.3(13) Checking Polyester Concrete for Bond

6-23.3(13)A Sounding

After the requirements for curing have been met, the entire overlay surface shall be inspected by the Contractor's independent testing entity, in the presence of the Engineer, in accordance with ASTM D4580, Method B. Any areas of delamination shall be removed and replaced at no additional expense to the Contracting Agency. Extensive unbonded areas may be grounds for rejection of the entire installation if ordered by the Engineer.

6-23.3(13)B Direct Tension Bond Testing

Vertical axis adhesion tests shall be performed not more than 24 hours after the placement of the Polyester concrete overlay by an independent testing company, arranged by the Contractor, in accordance with ASTM C1583, cost to be included in polyester concrete Overlay Placement item. At a minimum, two adhesion tests, at randomly selected locations, shall be performed on the first bridge and Trial Overlay. For bridges with deck areas greater than 25,000 square feet, or multiple bridge projects, additional tests shall be performed at a frequency of one test per 25,000 square feet of additional deck area, if required by the Engineer. If substrate and surface preparation remain consistent and sufficient, a single test set may be sufficient and subsequent tests may be waived if allowed by the Engineer. Additional testing may be required as directed by the Engineer if any element of the substrate, surface prep, polyester concrete overlay system, or placement changes after initial testing.

Test cores shall be drilled a minimum of 0.25" but no greater than 0.50" below the substrate to overlay bond line.

The minimum bond strength of the polyester concrete overlay system on normal weight concrete shall be 250 psi. An acceptable test will demonstrate that the overlay bond strength is sufficient by producing a concrete subsurface failure area greater than 50% of the test surface area ("type a" per test method). Failure at the epoxy/overlay interface ("type d" per test method) is also acceptable provided the failure occurs at not less than 250 psi. The Contractor shall repair all bond test locations with polyester concrete overlay in accordance with this Specification.

6-23.3(14) Crack Sealing Polyester Concrete

If cracks appear in the overlay after a significant cure period, they shall be filled with properly catalyzed and mixed HMWM primer material. Care shall be taken to fill the cracks only, and ensure minimal primer is left on the finished surface of the overlay.

If cracking is extensive, yet no other defects exist, the area shall be shot blast cleaned and flood coated with properly catalyzed and mixed crack sealer followed by broadcasting sand meeting the requirements for sand for abrasive finish.

6-23.3(15) Surface Smoothness

After crack sealing is complete, the Contractor shall test the entire deck/slab for flatness (allowing for crown, camber, and vertical curvature). The testing shall be done with a 10-foot straightedge held on the surface. The straightedge shall be advanced in successive positions parallel to the centerline, moving not more than one half the length of the straightedge each time it advances. This procedure shall be repeated with the straightedge held perpendicular to the centerline. An acceptable surface shall be both (1) free from deviations of more than ⅛-inch under the 10-foot straightedge, and (2) free from cyclical/repetitive vertical deviations greater than 1/16".

If smoothness testing identifies areas that deviate from the smoothness requirements, the Contractor shall grind these down with a diamond grinder meeting the requirements of Section 6-23.3(2)G. Prior to diamond grinding, areas showing low spots of more than ¼ inch in 10 feet shall be marked and prepared with shot blasting or sandblasting, primed, and filled with either catalyzed resin and broadcast sand or mixed polyester concrete slurry material if ordered by the Engineer. The use of resin or mixed slurry material shall be as recommended by the System Provider Technical Representative and approved by the Engineer. Grinding removal of the fill area boundary may be required if directed by the Engineer. Retesting and refinishing shall continue until a surface conforming to the requirements specified above is produced. The grinding depth of high areas after initial finishing shall not exceed ¼ inch.

6-23.3(16) Texturing Polyester Concrete

After the Contractor has completed all work required to meet the requirements for surface smoothness, the polyester concrete overlay surface shall receive a longitudinally sawn texture using equipment as described in Section 6-23.3(2)H. The Contractor shall texture the bridge deck surface to within 3-inches minimum and 12-inches maximum of the edge of concrete at expansion joints, within 1-foot minimum and 2-feet maximum of the curb line, and within 3-inches minimum and 9-inches maximum of the perimeter of bridge drain assemblies.

The Contractor shall contain and collect all concrete dust and debris generated by the bridge deck texturing process and shall dispose of the collected concrete dust and debris in accordance with its Debris Containment and Disposal Plan.

After texturing polyester concrete surface, the Engineer shall test the surface texture of polyester concrete for uniformity and it shall have a skid number (SN) of not less than 35 as determined by ASTM E 274.

6-23.3(17) Replacement of Defective Overlay

A defective overlay, or portion thereof, as evidenced by insufficient strength, lack of sound bond to substrate, or failing overlay adhesion test results shall be removed and replaced at the Contractor’s expense. The Contractor shall submit a written corrective action plan to the Engineer, which shall include the methods and procedures that will be used. The Contractor shall not commence corrective work until the methods and procedures have been approved in writing by the Engineer. The Engineer's approval shall not relieve the Contractor of the responsibility of producing work in conformity with the Contract.

6-23.3(18) Opening to Traffic

Prior to opening the overlay area to vehicular traffic, the finished overlay shall be power swept to remove excess loose aggregate and loose sand for abrasive finish. The Contractor shall demonstrate to the satisfaction of the Engineer that the power broom equipment will not damage the finished overlay. Damage to the finished overlay caused by the power broom shall be repaired at no additional expense to the Contracting Agency.

6-23.4 Measurement

Shotblasting concrete surface will be measured by the square yard of surface shotblasted.

Type 1 Deck Repair and Type 2 Deck Repair will be measured by the square foot of surface area of deck concrete removed in accordance with Section 6-23.3(7). Determination of whether a deck repair is Type 1 or Type 2 shall be in accordance with Section 6-23.3(7).

Polyester concrete overlay will be measured by the square yard of overlay surface actually placed.

6-23.5 Payment

Payment will be made for each of the following Bid Items that are included in the Bid Proposal:

"Surveying for Polyester Concrete Overlay", lump sum.

The lump sum contract price for "Surveying for Polyester Concrete Overlay" shall be full pay to perform the Work as specified, including establishing secondary survey control points, performing survey quality control, and recording, compiling, and submitting the survey records to the Engineer, and all other surveying required to complete the polyester concrete overlay.

"Type 1 Deck Repair", per square foot.

The unit contract price per square foot for Type 1 Deck Repair shall be full pay for performing the Work as specified, including excavating and disposing concrete and nonconcrete materials, and repair of concrete or rebar damaged by the Contractor's operations.

"Type 2 Deck Repair", per square foot.

The unit contract price per square foot for Type 2 Deck Repair shall be full pay for performing the Work as specified, including: excavating and disposing concrete; sandblasting; placing, consolidating, finishing, and curing concrete patches in Type 2 deck repairs; repair of concrete or rebar damaged by the Contractor's operations.

"Polyester Concrete Trial Overlay", lump sum.

The lump sum contract price for "Polyester Concrete Trial Overlay" shall be full pay for performing the Work as specified, including establishing a location for the trial overlay, construction, removal, and disposal of the concrete pad and trial overlay.

"Polyester Concrete Overlay", per square yard.

The unit contract price per square yard for "Polyester Concrete Overlay" shall be full pay for performing the Work as specified, including dry run, initial surface preparation, final surface preparation, placing primer, placing, finishing, and curing the overlay, placing sand for abrasive finish, sounding, direct tension bond testing, meeting surface smoothness requirements, texturing, crack sealing, and replacement of defective overlay. Polyester concrete overlay placed in excess of the thickness specified in the Plans due to surface irregularities in the bridge deck such as rutting or excess concrete surface shotblasting shall be considered incidental to the unit Contract price per square yard for "Polyester Concrete Overlay".

Payment for the following shall be considered incidental to and included in the unit contract items included in the Contract:

1. All Work and related costs for implementing the debris containment and disposal plan.
2. All Work and related costs for protecting adjacent traffic from flying debris.
3. All Work and related costs for managing and disposing of process wastewater.
4. Submittals.