Chapter 6 Structures

6-01 General Requirements for Structures

GEN 6-01.1 Bridge Construction De-Briefing Session

In an attempt to continually improve the quality of bridge contract plans, specifications and estimates and to obtain feedback on engineering and construction practices, the Bridge and Structures Office is available to assist in conducting post construction De Briefing Sessions for "Capturing Lessons Learned." The purpose of these De Briefing Sessions is to provide designers with feedback on positive things that worked well and things that could be improved.

The Project Engineer, Bridge Technical Advisor, or Bridge Design Unit Manager should consider initiating a De-Briefing Session on those projects where they feel feedback to the designers would benefit the quality of future construction plans. Suggested projects include Bridge Rehabilitation Projects, Bridges with complex staging, substructure conditions, or new material applications. Suggested attendees at these sessions should include Region Project Office Staff, State Construction Office, Bridge and Structures Office, Design Consultants, and the Contractor involved in the structural work.

The Bridge and Structures Office will assist the Project Engineer in organizing and facilitating the De-Briefing Session once it is agreed to go forward with a De-Briefing Session. The Project Engineer will be responsible for making all contacts with Contractor personnel.

The Project Engineer should determine the timing of the De-Brief session with respect to the contract work. Scheduling the session too long after the contract work is complete may diminish the Contractor's willingness to participate or recall of the issues for discussion. Scheduling a session too soon before completion of all contract related activities may cloud issues currently under discussion. The Project Engineer should exercise caution in selecting the proper timeframe for this session.

GEN 6-01.2 General Inspection Procedures

The intent of the Contracting Agency inspection is to provide Quality Assurance (Q/A) for the work performed. Often this task creeps into the Quality Control (Q/C) function which is the contractor's responsibility. There is usually no need for an inspector to observe the entire construction operation unless there are compelling reasons.

Because of the wide variety of types and designs of structures, the Project Inspector should be thoroughly familiar with all of the Contract documents as they provide the specific materials requirements, dimensions, and other details that make each structure unique.

Set up part of the inspection documentation records in advance so that the actual dates, dimensions, quantities, and other values can be more easily filled in as the work progresses.

GEN 6-01.4 Safety Nets and Staging

Fall arrest and protection shall be provided. Reference WAC 296-880 *Fall restraint, fall arrest systems*. A Fall Protection Work Plan shall be on site.

Standard Specifications Section 1-05.6 requires the Contractor to furnish sufficient, safe, and proper facilities such as walkways, railings, ladders, and platforms for inspection of the work. The Project Engineer should insist that the Contractor provide safe facilities and should not permit WSDOT personnel on the project when it is not safe for them.

SS 6-01.2 Foundations Data

Elevations of bottom of footings, as shown in the plans are determined from the information secured from test holes, borings or other sources. The Project Engineer shall observe the character of the materials removed to confirm that these materials are similar to those identified in the test borings. If the materials are similar, the Project Engineer shall note the elevation from where these materials were taken and approve the footing elevation. If the materials differ from the test borings, the State Construction Office shall be consulted for an evaluation.

Even in solid rock foundations it is necessary to construct the top of the footing below the line of scour to prevent future exposure of the substructure.

Footings on solid rock shall be keyed into the rock to prevent sliding. Keys should not be less than 1 foot deep and the rock surface shall be roughened.

Arch abutments may be designed with bottoms on an inclined plane. Care shall be taken that the rock or other materials are cut as nearly as possible to the plane shown in the Plans. If this cannot be done, the materials should be removed to a satisfactory elevation. The State Construction Office shall be advised and a request to secure a new design of the abutment shall be initiated. Materials at the heel of the abutment shall be carefully removed with all the loose materials present. When placing arch concrete abutments, the concrete shall be directly against the undisturbed foundation materials.

Footings in hard materials are sometimes sloped or stepped. Steps must be carefully constructed and if the materials composing the soil are not hard enough to stand vertically the steps shall be inclined or beveled. The slope shall not be steeper than the angle of repose. Backfilling to level up foundations or to fill holes will not be allowed except by permission of the State Construction Office under consultation with the State Geotechnical Office.

SS 6-01.4 Appearance of Structures

Bridge traffic barriers, curbs, bridge railings and rail bases shall be carefully aligned to give a pleasing appearance.

SS 6-01.6 Load Restrictions on Bridges Under Construction

It is important that bridges under construction remain closed to all traffic, construction equipment, and material storage (that will not become part of the bridge span) until the Substructure and the Superstructure, through the bridge deck, are complete for the entire Structure. The Contractor may request to allow traffic, construction equipment and material loads (in addition to those that will become part of the bridge span) if it is necessary and safe to do so through a Type 2E Working Drawing. See the *Standard Specification* for the specific submittal requirements. 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 the *Standard 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. The reasoning for not allowing materials in the middle third of the span is to avoid overstressing girders. They do not have full capacity until the bridge deck gains strength and becomes composite with the girders.

SS 6-01.9 Working Drawings

The Contractor is required to submit for review detailed plans for falsework, concrete forms, cofferdams, shoring, and cribbing. These plans must comply with the requirements of the contract plans and specifications and shall be designed under the supervision of or by a Washington State licensed professional engineer and shall bear their seal and signature.

The Project Engineer should review the submittal, when appropriate, for the following content:

1. Ground line at time of construction when falsework, shoring, and cribbing are involved.
2. Horizontal clearances to adjacent roadways, existing structures, and railroads when shoring and cribbing are involved.

A change order is required for any deviation from the contract. Deviation from a working drawing requires Headquarters' review and concurrence. Review of these submittals must be completed before the Contractor starts construction of the structure.

If a project has a large number of working drawings associated with it the Project Engineer should talk to the contractor about prioritizing his submittals. The project engineer should share this information with the State Bridge and Structures Engineer so that the review process can be accomplished in the most efficient manner for the contractor.

The Contractor shall submit drawings per the contract and Section SS 1-05.3 of this manual.

The Project Engineer will review the plans to see that they comply with the submittal requirements of the contract and send any comments to the State Bridge and Structures Engineer (or Terminal Design Engineer) about any field conditions or contract deficiencies that would affect the checking of the plans.

When pre-contract reviewed formwork plans are used, the Contractor shall submit a copy of the plans to the Project Engineer. The Project Engineer must then advise the Contractor that construction may proceed unless a field condition needs to be resolved before doing so.

Forms for concrete deck on steel or prestressed concrete girder spans shall be fully supported on the girders. They shall in no case extend to the ground unless the steel girders are also supported on piles or posts.

The Project Engineer shall see that the falsework and forms are constructed in accordance with the submitted plans. If it becomes necessary, or the Contractor desires to deviate from the submitted plans, a revised plan for review shall be submitted and the Contractor shall not start construction in accordance with the revised plan until the review is complete. All revisions to the plan shall be reviewed by the State Bridge and Structures Engineer (or Terminal Design Engineer) to ensure the structural integrity of the falsework and formwork.

SS 6-01.12 Final Cleanup

When the structure is completed, the Contractor shall clean up the site and remove all materials and debris. The decks of the structures shall be clean. The Contractor shall level off and fine grade all excavated material not used for backfill, and fine grade around all piers, bents, abutments, and on slopes so that the entire site and structure is left in a clean and presentable condition.

Unless environmental permits require otherwise, remove all falsework piling, cofferdams, shoring, curbs, and test piles to a minimum of 2 feet below the finished ground line. Removal limits within a stream or channel are described in *Standard Specifications* Section 3-07.3(3)D.

After a permanent or temporary bridge or a bridge modification is complete and preferably before opened to traffic, the State Bridge and Structures Office's Bridge Preservation Section needs to perform an inventory inspection. The purpose of this inspection is to field verify certain contract plan details, to provide a base-line condition assessment of the bridge, and to identify any potential problem features.

When the bridge is nearing completion, two to four weeks before completion, the Project Engineer should notify the State Bridge Preservation Engineer of the anticipated completion date. The Bridge Preservation Engineer will make arrangements with the Project Engineer for an inventory inspection.

SS 6-01.16 Repair of Defective Work

The purpose of this section is to contractually allow structural repairs without requiring a change order and to define requirements for structural repairs. It is not intended to overwrite or duplicate submittal requirements or require submittals for repairs described elsewhere in the Contract Documents.

The WSDOT Project Engineer shall consult with the ASCE and an appropriate licensed professional engineer (such as the engineer-of-record, the Bridge Technical Advisor (BTA), the State Bridge Construction Engineer, etc.) to make a determination of whether a repair procedure that is not pre-approved requires engineering as well as whether a pre-approved repair procedure is appropriate for use for the intended repair.

Pre-approved repair procedures for precast and prestressed concrete plants are located in their annual approval document. They are reviewed and approved by the State Construction Office. The process is described in the WSDOT *Materials Manual*, Standard Practice QC 6 and QC 7.

Working drawing submittals for repairs are primarily intended to provide the Engineer an opportunity to review and comment on repair procedures, facilitate proper inspection of the repair work, provide documentation of the repair, and assist the Engineer in preparation of the as-builts. All repairs shall be documented in the as-builts.

When construction issues at precast/prestressed concrete plants and steel fabrication plants need to be expedited, the fabricator may prepare a problem resolution form describing the problem and proposed resolution. The fabricator notifies the WSDOT Fabrication Inspection Office and receives their concurrence the problem has been accurately described on the Problem Resolution document. The concurrence is noted on the problem resolution form. The document is then emailed to both the Contractor (the Contractor forwards this on to the Project Engineer) and to the WSDOT Construction Office. The email addresses "structuralsteelprr@wsdot.wa.gov" for steel structures and "precastprr@wsdot.wa.gov" for precast concrete structures distribute to all of the WSDOT Construction Engineers and to the WSDOT Seattle Inspection Office. The WSDOT Construction Office reviews the document and prepares a recommendation for the Project Engineer. The WSDOT Project Engineer and the WSDOT Construction Office work together to address the fabricators proposed problem resolution. The Project Engineer will send the approval (or disapproval) to the Contractor and the WSDOT Fabrication Inspection Office.

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

This pre-approved repair procedure requires the Engineer to make a determination of whether the intended repair may affect structural adequacy. The Project Engineer shall consult with the ASCE and an appropriate licensed professional engineer (such as the engineer-of-record, the Bridge Technical Advisor (BTA), the State Bridge Construction Engineer, etc.) to make this determination.

Repairs that may be considered to affect structural adequacy include but are not limited to:

- Areas that extend deeper than the outer layer of reinforcement in members (or portions of members) that are or will be in compression such as columns, walls and portions of beams. Note that many repairs in compression areas will be able to be effective over time as the original un-damaged concrete creeps and transfers compression to the repair. This is especially true for high strength, low shrinkage repair materials.
- Areas in concrete that are already loaded by subsequent actions such as prestressing, release of falsework, subsequent material placement, or applied earth pressure
- Areas with significant reinforcing steel damage, corrosion or section loss.
- Areas with significant overhead work
- Areas that have been previously repaired
- Areas adjacent to post-tensioning anchorages
- Areas with numerous or large spalls in the concrete surface

The full extent of the damage may not be known until the damaged concrete is removed. For this reason the Contractor is directed to stop work after initial concrete removal. The Project Engineer may require the Contractor to submit a modified repair procedure. This may be appropriate when the area or volume of concrete is significantly greater than originally estimated or reinforcement/embedments are damaged or displaced. Other unforeseen conditions may also arise which may bring the validity of the pre-approved repair procedure into question. The Project Engineer should consult with the ASCE and appropriate licensed professional if it is suspected that the pre-approved repair is no longer appropriate. The Project Engineer can then require a revised repair procedure be submitted by the Contractor.

Shrinkage-compensating repair materials are made with an expansive cement or expansive component system in which initial expansion, if properly restrained, offsets strains caused by drying shrinkage. Shrinkage-compensating repair materials may not be appropriate if the repair area will not sufficiently restrain the initial expansion of the repair material with forms, surrounding concrete and reinforcement passing through the repair area.

6-02 Concrete Structures

GEN 6-02.1 Use of Epoxy Resins

Quite frequently, the use of epoxy resin systems on our projects is considered; either at the design stage or during the progress of a contract. Generally this use is in connection with repair of distressed concrete or in setting rebar.

Epoxy resins are quite versatile materials and are capable of providing the answer to numerous bonding or grouting problems. However, like a number of products, there is a tendency to treat them as a universal cure-all and they occasionally are applied without proper consideration of inherent limitations.

Epoxy systems are capable of providing many different properties through the formulation of their various components. To a certain extent, the systems can be tailored to fit the particular need and conditions of time, temperature, humidity, etc., that will prevail. Use of a material under conditions beyond those for which it was formulated can result in considerable trouble rather than benefit. Probably the greatest potential for trouble exists in the use of epoxies at temperatures below which a normal reaction can occur. Generally speaking, unless a specially formulated epoxy is being used, trouble can occur when application is attempted below 50°F.

The State Materials Engineer is available as a technical resource on the use of such systems, in the resolution of pertinent problems should they occur during preliminary design considerations, or as a result of problems during construction. It is strongly recommended that any contemplated use of epoxy resin systems at application temperatures below 50°F be checked with the State Chemical Materials Engineer to forestall potential difficulties.

If epoxy resin is used, the following elements need to be carefully checked by the Inspector:

- Proper mixing and curing of the epoxy resin.
- Temperature and/or moisture limitations of the epoxy being used.
- That the areas are clean and prepared in accordance with the manufacturer's recommendations.
- That the epoxy covers the entire repair area.
- That the epoxy fills the entire space between bar and the hole (if bars are being set with epoxy resin).
- That the epoxy is still tacky (not set) when it is being used to bond two structural elements together (just before elements are put together).

For setting rebar or anchors, it is best to determine the volume required to be filled by the epoxy and measure the epoxy being used. A method of measurement should be agreed to with the Contractor for inspection purposes. Also, occasional samples should be taken of the epoxy resin being placed to be sure it is setting up properly. If there is any question of filling the void or adequacy of the epoxy resin, the Inspector shall advise the Contractor, document the discussion, and report it to the Project Engineer.

SS 6-02.3 Construction Requirements

SS 6-02.3(2) Proportioning Materials

Mix design, proportioning, and mixing concrete is the responsibility of the Contractor. General information regarding proportioning and mixing concrete is provided in Appendix A at the end of this chapter to provide a better understanding of the variables involved.

SS 6-02.3(2)A Contractor Mix Designs

The *Standard Specifications* require the Contractor to provide a mix design for all classes of concrete specified in the Plans except for those accepted based on a Certificate of Compliance. The mix design should be submitted on Proposed Mix Design (DOT Form 350-040). The same concrete Mix Design No. may be used in several of a concrete suppliers Plants. Note that a unique identification for the mix design is comprised of the combination of the Mix Design Number and the Plant Number. The average 28-day compressive strength shall be selected in accordance with ACI 301, Chapter 4, Section 4.2.3.3 and ACI 201.2 shall be used to determine proportions. The Project Engineer should review all Contractor proposed mix designs for conformance to the contract. Specific items to look for are:

- Total water soluble or acid soluble chloride ion content
  - Verify the water soluble or acid soluble chloride ion content complies with *Standard Specifications* Section 6-02.3(2).
- Cementitious materials (Portland Cement, Low Alkali Cement, Blended Hydraulic Cement, Fly Ash, Ground Granulated Blast Furnace Slag, Microsilica Fume, and Metakaolin)
  - Verify the products are list on the QPL or have been approved through the RAM process.
  - Verify the type of cement is allowed by the Contract.
  - Check that mill certification demonstrates specification compliance.
  - Verify the proposed quantities within specification limits for the concrete class.
- Aggregate (Coarse, Fine, and Combined Aggregate)
  - Ensure the aggregate is from an approved source by verification of the ASA database.
  - Check if ASR mitigation is required by verifying the ASA database.
  - Verify the mix design submittal includes data for Deleterious Substances.
  - Ensure the Nominal Maximum Aggregate Size (NMS) is correct for the proposed concrete class.
  - Verify the proposed gradations meet the requirements of the concrete class.
  - Make sure the mix design indicates the quantities of aggregate.
- Alkali Silica Reactivity (ASR)
  - If the aggregate source is ASR reactive, verify the Contractor provided mitigation measures.
  - Ensure the mitigation measures demonstrate compliance with *Standard Specifications* Section 9-03.1(1).

- Admixtures
  - Verify products are listed on the QPL or have been approved through the RAM process.
  - Ensure the proposed quantities are within manufacturer's recommendations.
  - Verify all admixtures are from the same manufacturer.
- Water
  - Ensure the quantity of water is indicated on the mix design.
  - Verify the maximum water/cementitious ratio provided is equal to the total water divided by the total cementitious materials indicated on the mix design.
  - Ensure the full amount of water specified in the mix-design is in the test sample.
  - If reclaimed water is proposed, verify that it complies with *Standard Specifications* Section 9-25.1.
- Design Performance (applies to all concrete classes)
  - Compressive Strength
    - Ensure the break data and ACI equations supporting the concrete are provided with the mix design.
    - Verify the calculated average compressive strength meet the requirements for the concrete class.
  - Air Content
    - Verify the mix design indicates entrained air content between 4.5 – 7.5 percent. This criterion does not apply to concrete Class 4000D.
- Design Performance Concrete Class 4000D (additional requirements)
  - Permeability, AASHTO T 277.
    - Verify the mix design indicates a permeability of 2,000 coulombs or less at 56 days.
  - Freeze-thaw Durability
    - Verify the mix design indicates an air content between 4.5 – 7.5 percent, or
    - Resistances of Concrete to Rapid Freezing and Thawing, AASHTO T 161 Procedure A.
    - Verify the mix design indicates a durability factor of 90 percent minimum, after 300 cycles.
    - Verify the mix design indicates an air content equal to or greater than 3.0 percent.
  - Scaling Resistances of Concrete Surfaces Exposed to Deicing Chemicals, ASTM C 672.
    - Verify the mix design indicates a scaling visual rating less than or equal to 2 after 50 cycles.
  - Length Change of Hardened Hydraulic Cement Mortar and Concrete, AASHTO T 160.
    - Verify the mix design indicates a length change (shrinkage) at 20 days, less than or equal to 0.032 percent.
  - Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete, ASTM C 138.

- Design Performance Self-Consolidating Concrete (additional requirements).
  - Slump Flow
    - Ensure the mix design includes the targeted slump flow (WSDOT FOP for ASTM C 1611).
    - Verify the mix design indicates a Visual Stability Index (VSI) less than or equal to 1 (Appendix X1 of ASTM C 1611)
    - Verify the mix design indicates a T50 flow rate less than or equal to 6 seconds. (Appendix X1 of ASTM C 1611).
  - Column Segregation
    - Verify the mix design indicates a Maximum Static Segregation less than or equal to 10 percent (ASTM C 1610).
    - Verify the mix design indicates a Maximum Hardened Visual Stability Index (HVSI) less than equal to 1 (AASHTO PP 58).
  - Passing Ability of Self-Consolidating Concrete by J Ring, WSDOT FOP for ASTM C 1621.
    - Verify the mix design indicates J Ring results equal to or less than 1.5 inches.
  - Rapid Assessment of Static Segregation Resistance of Self-Consolidating Concrete Using Penetration Test, ASTM C 1712.
    - Verify the mix design indicates a penetration depth equal to or less than 15 millimeter.
  - Air Content of Freshly Mixed Self-Compacting Concrete by Pressure Method, WSDOT Test Method T 818.
    - Verify the mix design indicates entrained air content between 4.5 – 7.5 percent.
  - Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete, AASHTO T 121.
    - Ensure the mix design includes the unit weight (lbs/ft^3).
  - Temperature of Freshly Mixed Portland Cement Concrete, AASHTO T 309.
    - Ensure the mix design includes the temperature of the freshly mixed concrete

Air-entrained concrete is required all cast-in-place structural concrete above ground. The use of air entrained concrete below the finished ground line is optional with the Contractor.

The State Materials Laboratory has developed a mix design checklist located on the Construction SharePoint site. Contact the State Materials Laboratory for assistance reviewing concrete mix designs.

SS 6-02.3(4) Ready Mix Concrete

SS 6-02.3(4)A Qualification of Concrete Suppliers

All concrete production facilities which produce concrete other than commercial concrete or lean concrete will be prequalified. Commercial concrete and lean concrete may be batched in production facilities which are not prequalified. The concrete production facility prequalification requires 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 two year period from the date of certification.

The Contractor is required to submit Request for Approval of Materials Source (Form 350-071) listing the name and location of the plant which will supply the concrete and also the source of the cement, aggregates, and admixtures that will be used in the concrete. Concrete from the plant shall not be used until the plant has been approved. The Project Engineer shall take approval action based upon the batch plant prequalification submittal meeting the requirements of the Standard the Approved Source of Material Listing. If the batch plant prequalification submittal indicates that the scale certification has expired the Project Engineer shall confirm that the scales have been recertified or the source will not be approved.

Whenever ready mix concrete is used on the project, the Inspector shall be alert to the condition of the trucks being used for delivery. All trucks used for delivery of concrete (other than commercial concrete or lean concrete) must be preapproved prior to use on the project. Preapproval of delivery trucks is a part of the plant approval process described in Section 6-2.2A. Approved trucks will be identified on an NRMCA truck list for plant manager inspected facilities. Approved trucks will be identified by an NRMCA sticker (for the years of approval) for NRMCA approved facilities. In some cases an approved truck may not have yet received an NRMCA sticker. In these cases, the ready-mix producer shall notify the Project Engineer in writing that the truck has passed NRMCA inspection, and is approved for use. The Inspector should verify that all delivery trucks meet the requirements of Standard All delivery must have operational revolution counters and a device to measure the amount of water added at the site. All trucks are required to be operated within the rated capacity stated on the manufacturer's data plate. The Inspector needs to check the concrete as it is being discharged down the chute to ensure that the concrete is uniformly mixed. If the concrete does not appear uniformly mixed, the Inspector can request that the concrete producer re-inspect the truck. If the concrete delivery truck cannot deliver uniformly mixed concrete, the delivery truck needs to be rejected.

When necessary, the Project Office shall make an inspection of the batch plant to confirm: the accuracy of the batching process; that the scales have current certifications; the accuracy of the water metering devices; and to sample the coarse aggregate and fine aggregate.

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

The purpose of upper temperature limits of concrete for placement is to limit the ultimate temperature of the concrete reached during cement hydration and curing. This in turn tends to reduce the thermal differential between the ambient environment and the concrete. The reduction in the thermal differential helps reduce cracking of the concrete by limiting tensile strains. Cracking, particularly in decks, reduces the durability of the concrete and reinforcement.

Some techniques that concrete producers can use to meet the upper temperature for placement limit for concrete include:

- using minimum cement content,
- using pozzolans (such as fly ash) to replace a portion of the cement,
- using water-reducing admixtures,
- using air-entrainment,

- using large aggregate,
- shielding aggregate piles from direct sunlight,
- using cold water or chipped ice for mixing water,
- using liquid nitrogen.
- scheduling work for cooler times of day.
- shielding ready-mix trucks on-site from sunlight.
- spraying water on drums and pump lines.

While ideally the temperature of Class 4000D concrete for placement will be below 75 degrees F, it may be difficult for concrete producers to achieve this, especially during extended periods of high temperature. The specification allows the Contractor to ecxeed the 75 degree F upper limit for placement, up to a maximum of 80 degrees F, but the higher temperature limit requires a shorter time to discharge. All other classes of structural concrete can be accepted with a placement temperature up to a maximum of 90 degrees F with a discharge time of 1.5 hours.

The Project Engineer may extend the time to discharge by 15 minutes, provided the temperatures are within limits and the Contractor has requested an extension. The intent is to not reject an entire truck because of a delay in the haul. This extension is not intended to be granted to all trucks prior to the start of work, it is intended to accommodate contingencies that would otherwise risk a cold joint or unnecessary rejection of good concrete.

When transport times from the nearest viable supplier require more than a 15 minute extension of time, the Contractor may submit a Type 3 Working Drawing to request additional time to discharge. The time extension and mix design must be coordinated. The mix design must be designed with appropriate admixtures to enable increased haul times. Discharge times of more than 3 hours will not be approved.

Note, there may be instances when the Project Engineer may need to shorten the maximum time to discharge listed in the Standard Specifications. This is especially important for concrete that may experience an accelerated initial set. This most commonly occurs when accelerating admixtures are used or temperatures are elevated.

SS 6-02.3(5) Acceptance Concrete

The Contractor is required to provide a certificate of compliance for each load of concrete delivered to the job. Based on who is supplying the mix, the format of the certification may vary. All certifications must contain the information required by the *Standard Specifications*. If a Contractor Certification sheet is not provided by the Contractor, the form provided by WSDOT may be used. Example forms are available as follows:

- Manufacturer's Certificate of Compliance for Ready Mix Concrete (DOT Form 450-001)
- Proposed Mix Design (DOT Form 350-040)

A Certificate of Compliance is all that will be required for acceptance of commercial and lean concrete. It is advised that as inspectors are collecting the Certificate of Compliance (batch ticket), they do a visual inspection of the concrete. Visual inspection should verify that the items listed on the batch ticket are included in the mix. If the concrete does not appear satisfactory for its intended use, it should be rejected.

SS 6-02.3(5)C Conformance to Mix Design

It is the responsibility of the Inspector to compare the actual batch weights on the concrete delivery ticket to the proposed mix design weights. The cement, coarse and fine aggregate weights are required to meet the following tolerances:

Concrete batch volumes less than or equal to 4 cubic yards:

Cement	+5 percent and -1 percent
Aggregate	+10 percent and -2 percent

Concrete batch volumes greater than 4 cubic yards:

Cement	+5 percent and -1 percent
Aggregate	+2 percent and -2 percent

If the total cementitious material weight is made up of different components, the component weights shall be within the following tolerances of the amount specified in the mix design:

Portland cement weight	+5 percent and -1 percent
Fly ash weight	+5 percent and -5 percent
Microsilica weight	+10 percent and -10 percent

For all mix designs the water weight shall not exceed the maximum water specified in the mix design. These batching tolerances apply to all mixes.

SS 6-02.3(5)D Test Methods

Acceptance testing will be performed by WSDOT in accordance with WSDOT standard test methods and Field Operating Procedures. Lean concrete and commercial concrete will be accepted based on a Certificate of Compliance, provided by the supplier as described in *Standard Specifications* Section 6-02.3(5)B. All other concrete will be accepted based on conformance to the requirements for temperature, slump, air content for concrete placed above finished ground line, and the specified compressive strength at 28 days.

The Inspector must be familiar with the type of concrete mix and who is responsible for the mix. The Contractor is responsible for the mix design and is responsible for 28 day strength.

The Inspector must be prepared to test materials for conformance. The Inspector must also be prepared to deal with nonconformance.

Preparation as a concrete testing inspector requires knowledge of concrete properties and construction procedures. Knowledge of how to use testing equipment and understanding the reliability of testing is also important. A continual evaluation of the testing equipment is needed to be sure it is operating and performing as required. Care and caution are recommended when transporting testing equipment and handling test materials, i.e., cylinders, molds, fresh concrete cylinders, and other samples).

The maximum slump for vibrated and nonvibrated concrete is listed in *Standard Specifications* Section 6-02.3(4)C.

When a high range water reducer (super plasticizer) is used, the maximum slump limit may be increased an additional 2 in while the concrete is affected by the admixture.

All cast-in-place concrete above the finished ground line shall be air entrained. The air content shall be a minimum of 4.5 percent and a maximum of 7.5 percent, unless otherwise specified.

When commercial concrete is placed in sidewalks, curbs, and gutters, air content is very important. It is recommended that the inspector perform air content testing sufficient to ensure that the concrete has between 4.5 and 7.5 percent air entrainment.

The Contractor may elect to use air entrained concrete below finished ground line. If so, the 28-day compressive strength shall meet the requirements for the class of concrete specified.

It is the Inspector's job to ensure that:

- The concrete is placed in the forms as soon as possible after mixing, but no later than 1.5 hours after cement is added to the mix.
- The concrete is always plastic and workable while being placed.
- The concrete is placed continuously with interruptions no longer than 30 minutes.
- Each layer of concrete is placed and consolidated before the preceding layer takes initial set. Initial set has begun if the vibrator will not penetrate the preceding layer under its own weight while being operated.

The discharge time may be extended to 1.75 hours if the temperature of the concrete being placed is less than 75°F. With the approval of the Project Engineer, this may be extended to two hours, if the temperature of the concrete being placed is less than 75°F. If it is apparent that the 30-minute time limit will be exceeded for a continuous pour, a construction joint should be established. The State Construction Office shall be contacted when this occurs. A vibrator can be used to determine if initial set has taken place when evaluating the need for a construction joint as described previously.

In certain instances, it may be difficult to meet the above criteria due to long transit times. The *Standard Specifications* allow the Contractor the option of requesting in writing to extend the time for discharge. The extension of time will be considered on a case by case basis and requires the use of specific retardation admixtures and coordination with the State Construction Office.

SS 6-02.3(5)E Point of Acceptance

Acceptance tests for specification compliance are to be determined from samples taken at the discharge of the placement system for bridge decks, overlays, bridge approach slabs, and barriers, and at the truck discharge for all other placement. For bridge decks, overlays, bridge roadway slabs, bridge approach slabs, and barriers, acceptance samples should be taken as close to the point of deposition as possible. (e.g., taking a sample from the end of a pump down below the bridge instead of up on the deck is not acceptable as it may have substantially different characteristics.)

If a pump is used as a placement system, the initial acceptance test must be delayed until the pump has been cleared of all initial priming slurry. Do not allow placement of pump slurry in the forms.

The Inspector should arrive in advance of the concrete placement and prepare the testing location. It is the Contractor's responsibility to provide adequate and representative samples of the fresh concrete to a location designated by the Engineer. Above all, the equipment must be in good working condition with records of the last calibrations for the air meter and scales. The Inspector should have all the information, including the mix design, and all the forms needed for documentation of the placement operation.

Concrete test cylinders shall be molded in forms conforming to the requirements for single use molds as detailed in ASTM M 205. Cardboard test cylinder molds shall not be used.

See Chapter 9 for instructions for making, curing, and shipping concrete test cylinders and for the number of test cylinders to be made.

Extra cylinders that are tested for early removal of forms and falsework shall be the responsibility of the Contractor. Early cylinders are cylinders tested in advance of the design age of 28 days. Their purpose is to determine the in place strength of concrete in a structure prior to applying loads or stresses. The Contractor shall retain an independent testing laboratory to perform this work. This lab shall be approved by the Engineer.

The cylinders shall be cured in accordance with WSDOT FOP for AASHTO R 100. Special cure boxes to enhance cylinder strength will not be allowed. The number of early cylinder breaks shall be in accordance with the Contractors need and as approved by the Engineer.

Prior to the removal of any forms, the Contractor is required to furnish the Engineer with all test results. Forms shall not be removed without approval of the Engineer.

If set retarders are used in a mix, the State Materials Lab should be consulted for curing, handling, and storage instructions prior to use.

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.

Only one set of acceptance tests are required per concrete truck.

SS 6-02.3(6) Placing Concrete

A Concrete Placement Checklist was developed as an inspection aid and is available on the Construction Office SharePoint site in the *Construction Manual* Resources folder.

If it is necessary or desirable to place structural concrete in service prior to the time stated in the *Standard Specifications*, authority must be obtained from the State Construction Office. In such cases, test cylinders from each pour are taken and tested by the Contractor to determine the early break strength.

All sawdust, nails, dirt, and other foreign material, including ponded water, must be removed from within the forms and the forms shall be inspected and approved before placing any concrete.

The bottom of footings and forms must be thoroughly soaked with water prior to placing the concrete so they do not absorb water from the concrete mix. Care must be taken to be sure there is no ponded water when placing the concrete.

Concrete in all reinforced footings shall be placed in the dry. All reinforcing, including vertical wall or shaft bars and dowels, shall be securely fastened in place before placing of concrete begins. Driving of dowel bars into concrete must not be permitted, except in seal concrete when the seal is also the footing block, but they must be placed immediately after the concrete is placed. The placing and spacing of footing reinforcing steel is as important as in any other part of the structure.

Care must be exercised in placing reinforcing steel in the columns where it splices with the dowel bars into the footings. In many instances, if the dowel bars and column bars are not carefully placed, there is not enough space between the steel bars for proper placement of concrete. Considerable care must be taken in placing and vibrating the concrete in the columns so that no rock pockets are formed. Column details must be strictly adhered to since they are critical to the earthquake resistance of the bridge.

Care must be taken in placing and vibrating the concrete of sloping walls or columns to get proper consolidation and to avoid rock pockets.

Concrete shall be placed in one continuous operation from top of footing to bottom of pier cap or crossbeam unless construction joints are shown in the plans or preapproved by the State Construction Office. Concrete shall be placed at the rate for which the formwork is designed. This rate, in ft of height per hour along with the concrete temperature, should be stated on the falsework plans. Spacing of studs, wales and form ties shall be as shown on the falsework plans. Rails, barriers, and parapets on retaining walls shall not be placed until all backfilling is completed. Vibrators shall be used at all times when placing concrete, unless otherwise specified.

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

Concrete may not be placed when rain is hard enough to:

- Cause a muddy foundation.
- Wash or flow the concrete.

The temperature of the concrete for cast-in-place concrete must be between 55°F and 90°F during placement. The temperature for precast concrete that is heat cured must be between 50°F and 90°F.

The air temperature must be at least 35°F during and for seven days after placement (unless the contractor has a cold weather plan in place).

The temperature measuring device shall be capable of measuring the temperature of freshly mixed concrete to ±1°F with a range of 0°F to 130°F.

SS 6-02.3(6)A1 Hot Weather Protection

- Cool the component materials of the mix, transport and placement equipment, and the contact surfaces at the site.
- Methods shall be reviewed prior to implementation.

When the concrete is being placed in the bridge deck during hot weather, additional precautions must be taken in order to prevent surface evaporation. See *Standard Specifications* Section 6-02.3(6)A for estimated evaporation rates.

The temperature of the concrete at the time it is placed in the forms must be kept under 90°F. Concrete with high temperature loses slump rapidly and is difficult to place and finish. This temperature can be controlled by shading the concrete trucks while loading and unloading and shading the conveyors or pump lines used in placing the concrete. The forms and reinforcing steel should be cooled prior to placing the concrete. This can be done by covering them with damp burlap and then spraying them with cool water immediately prior to placing the concrete. Care must be taken to see there is no standing water in the forms when the concrete is placed.

Water reducing retarder admixture should be used in the concrete so the water-cement ratio and slump of the concrete can be maintained within the specification limits. The mixing time of the concrete should be held to the minimum. The concrete must be placed and finished as soon as possible. If there is a delay in applying the curing compound after the concrete has been finished, a fog spray should be applied to reduce the moisture loss due to evaporation. If plastic cracks form and the concrete is still in a plastic state, they can be eliminated by revibrating the concrete and refinishing. Care must be taken to not revibrate the concrete after initial set has been obtained.

The requirements for curing the concrete shall be enforced. As soon as the visible bleed water has evaporated from the finished deck, the curing compound should be applied. The curing compound should be applied in two applications to ensure full coverage of the concrete. The second coat should be applied in a direction perpendicular to that of the first application. The amount of curing compound applied in the two applications should meet the minimum amount specified. Immediately after application of the curing compound and initial set, the concrete deck should be covered in accordance with *Standard Specifications* Section 6-02.3(11).

In summary, the difficulties arising from hot weather concreting may usually be minimized by:

1. Using cool mixing water.
2. Keeping the aggregate temperature as low as is economically feasible.
3. Reducing the length of mixing time.
4. Placing the concrete as soon as possible after mixing and with a minimum of handling.
5. Keeping the surfaces shaded during placing.
6. Placing curing compound as soon as possible.

SS 6-02.3(6)A2 Cold Weather Protection

- Concrete shall not be placed against any frozen or ice-coated foundation, forms, or reinforcement.
- A plan for cold weather placement and curing is required, if temperatures are below 35°F or anticipated to be below 35°F in the next seven days.
- Heat aggregate and/or water to maintain mix temperatures above 55°F.
- Control temperature and humidity after placement by:
  - Enclosing concrete.
  - Heating to 50°F to 90°F for seven days.
  - Add moisture for six days (discontinue 24 hours before heat is stopped).
  - An accurate recording thermometer is required.
  - Corners and edges require special attention to prevent freezing.

When heating water and aggregates, the approximate resulting temperature for a batch of concrete can be estimated from the following formula:

$$X = \frac{Wt + 0.22W't}{W + 0.22W'}$$

Where

X	=	temperature of the batch
W	=	weight of the water
W′	=	weight of the aggregates and cement
t	=	temperature of the water in degrees F
t′	=	temperature of the aggregates and cement

Several precautions must be taken when placing concrete in cold weather. If temperatures below 35°F are anticipated within seven days following placing the concrete, the Contractor will normally be required to enclose the structure and provide heat and moisture so the concrete will obtain its initial strength without freezing. The addition of moisture should be discontinued 24 hours before discontinuing the heat so there will not be an excess of moisture on the surface of the concrete to form ice in case of cold weather following the seven-day protection. If the temperature is below 35°F when placing the concrete, the concrete must be heated to at least 60°F by heating the aggregate and/or water in accordance with the *Standard Specifications*. The temperature of the concrete, as well as the slump, must be consistent from batch to batch.

When heating water and aggregates, the resulting temperature for a batch of concrete can be computed from the formula in Section 6-2.3A(1).

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

When constructing foundations in streams and other locations below water, it is usually necessary to place a concrete seal in the cofferdam so that the cofferdams may be dewatered. The weight of the concrete seal resists the buoyant force on the cofferdam when it is dewatered. Seal concrete is placed underwater by means of a tremie. Concrete pumps may be used.

Handling of the tremie requires the use of a crane to raise and lower it into place. Hand winches are sometimes used in small seals but they must be equipped with a brake and drum for quick release and stop.

The tremie pipe shall be at least 10 inch in diameter, made of heavy steel pipe, with flange or sleeve connections. Sleeve connections are preferable for seals placed in pile foundations. Flanges sometimes hang up on tops of piles and the concrete charge is lost. The tremie pipe must be absolutely water tight, at the joints as well as at the connections to the hopper. The hopper should be of at least, one-half cubic yard capacity.

Before any concrete is placed, the bottom of the tremie pipe shall be sealed with a plug. A satisfactory plug can be made with a 2-inch board slightly larger in diameter than the tremie pipe; on top of this board fasten a ¾-inch round piece cut to the neat size of the inside of the pipe. Place a piece of cloth or burlap over the end of the pipe and drive the plug in place. Lower the tremie until the plug rests on the bottom, then fill the tremie pipe with concrete. When the tremie is raised the weight of the concrete will push out the plug. The plug can be salvaged by fastening a piece of wire to it before it is lowered into the water.

Further details for handling a tremie are found in *Standard Specifications* Section 6-02.3(6)B.

The thickness of seals without piling are generally not less than 0.43 times the height of high water above the bottom of seal. Seals in footing with piling require special design. The thickness of the seal is computed for the water elevation shown in the plans. The cofferdams must be designed and vented for this elevation. The design and vent elevations are noted in the plans. If concrete is placed in the seal during a period of high water, the dewatering of the cofferdam will have to be delayed until the water level drops to the vented elevation. No change in the vent elevation shown in the plans shall be allowed without approval from the State Construction Office. Such approval should be obtained before the cofferdam is designed. All cofferdams must be vented at the elevation used for computing the seal thickness in order to prevent an unsafe hydrostatic pressure on the seal. Cofferdams shall not be dewatered before the concrete has been placed and cured.

The vertical sheathing of the cofferdam or shoring shall extend below the bottom of the excavation in accordance with the working drawings. Sheet piles in cofferdams shall be placed tightly together so that there will be no flow of water through the cofferdams while seal concrete is being placed.

The tops of seals should slope slightly toward one end. At that end, provision shall be made for a sump for the pump intake. Cofferdams should be tightly constructed so that a minimum of pumping is required after the cofferdam has been dewatered. Space for water courses shall be provided on top of the seal and around the footing block, between the footing block and the walls of the cofferdam.

Before starting to place seal concrete, all equipment should be checked to see that it is in good working order. It is necessary that concrete in a seal be placed continuously until completion, with the end of the tremie always extending into the fresh concrete.

It is not desirable to leave cofferdam struts and waling in the seal concrete but it is sometimes necessary to do so, especially in soft foundation material, when a set of struts and waling is required near the bottom of the cofferdam. The concrete displaced by such struts and waling is not deducted from the Contractor's pay items.

After the cofferdam is dewatered, a film of scum or laitance will usually be found on top of the seal. This must be cleaned off before the footing concrete is placed. If the seal is designed as a footing, the laitance will have to be removed only from the areas that will support pier shafts, columns, or walls.

SS 6-02.3(8) Vibration of Concrete

Vibrators are usually specified to be used when placing concrete. Their use is important for the purpose of consolidating the concrete in the forms, thus producing a dense uniform concrete.

Adequate vibration is necessary for placing concrete in difficult places, such as under and around closely spaced reinforcement. When steel forms are used for curbs, traffic barriers, or rail bases, external vibration may be required to eliminate voids at the surface caused by entrapped air. It is desirable to have the Contractor designate one person to operate the vibrator. This person could then be instructed in its use and an effort could be made to have that person kept on the same work whenever it is required.

The quantity of mixing water to be used shall be the minimum amount possible to produce the required workability. Vibrators shall be used only in freshly placed concrete. As soon as the concrete is dumped it should be spread out and vibrated by inserting the vibrator torpedo directly into the fresh concrete. However, it should be kept in one place only long enough to make the concrete uniformly plastic. Dependence should not be placed on the vibrator to work the concrete into corners and along the faces of the forms. Metal or wooden spades should be used to whatever extent is necessary in places where the vibrator cannot be satisfactorily employed, however, spades should be used only to accomplish complete filling of the forms and not for the purpose of puddling the concrete.

In regard to the desired consistency of concrete and the use of vibrators, the *Standard Specifications* should be carefully studied and followed. Every effort should be made to see that the specifications are followed.

Concrete shall be placed in accordance with the requirements of *Standard Specifications* Section 6-02.3(6). The Inspector should be alert to see that any method of placing concrete that causes segregation of the concrete mix be discontinued. Some of the conveyor belt systems tend to cause segregation of the mix after several exchanges from one belt to another. The Inspector shall see that the length of conveyor belt is limited so segregation does not occur. Aluminum pipe or sheeting shall not be used in contact with fresh concrete.

In heavily reinforced sections, the maximum concrete slump may be increased 2 inches with the use of a high range water reducer, as discussed in *Standard Specifications* Section 6-02.3(4)C. It is anticipated that possible candidates for this increase of concrete slump may be columns, cross-beams, and post-tensioned box girder web walls and other heavily reinforced members.

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

Bridge deck construction is critical because this part of the structure receives the most abuse from traffic and the environment. Construction of maintenance-free bridge decks requires close attention to details. One or two weeks before placing the concrete in the deck, a placement conference should be held to go over the procedures to be used and to emphasize the critical areas of construction. As a minimum, this should include a discussion of the rate of placement, personnel and equipment and backup equipment to be used, type of finish, and curing details. The rate of placement should normally provide for at least 20 feet of finished deck per hour.

The position of the reinforcing steel is very important because of the thin concrete section. Adequate blocking and ties are necessary to hold the steel in place. If foot traffic on the reinforcing steel causes it to deflect, the spacing of the chair supports is not adequate. A pre-check of the screed setting for proper elevations and clearances to the reinforcing steel is essential prior to any concrete placement. The finishing machine should be run the full length of the placement after the screed is adjusted to check deck thickness and cover of the reinforcing steel, this check should also continue over all bulkheads and expansion joints to verify their clearances. The finishing machine should not be adjusted while it is finishing concrete to clear bulkheads and expansion joints. These adjustments must be made prior to the concrete placement. During the placement, frequent checks should be made of the actual cover obtained directly behind the finishing machine and recorded in the Inspector's Daily Report.

Quality concrete is required, particularly in the bridge deck. Uniform consistency of the concrete should be maintained throughout the placement. The water-cement ratio is very important. It should be the minimum possible to produce the required workability and not exceed the specification limit. To keep the water-cement ratio as low as possible, the specifications require the use of a water reducing additive for all bridge deck concrete. Frequent checks of the free water contained in the aggregates is necessary to determine the amount of water actually contained in the concrete mix.

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

Construction of crack-free and maintenance-free bridge decks requires close attention to details during concrete placement and curing. One or two weeks before placing the concrete in the deck, a pre-deck pour meeting shall be held to go over the procedures to be used and to emphasize the critical areas of construction. Points of discussion should include concrete delivery and sampling, placement rates, personnel and equipment to be used, finishing, and curing details. The placement and operation of the temperature measuring and recording devices should also be discussed. The rate of placement should normally provide for at least 20 feet of finished deck per hour. Attendance at the pre-deck pour meeting should include:

1. Representing the Contractor, the superintendent, foremen in charge of placing and finishing concrete, a representative from the concrete supplier and the pump truck operator.
2. Representing WSDOT, the Project Engineer, Chief Inspector and key inspection and testing personnel. A representative from the State Construction Office should be invited.

A sample pre-deck pour meeting agenda for use by the Project Office can be found on the *Construction Manual* Resources website.

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

Finishing of roadway slab and bridge approach slab surfaces shall be as outlined in *Standard Specifications* Section 6-02.3(10). The principal objectives to be attained are a good wearing surface and a smooth riding roadway. The Engineer should ensure that adequate preparation has been made to do a good job in accordance with the specifications. The Engineer should insist that a float be available. When a good strike-off and finish has been obtained by a finishing machine, floating may be, and should be, kept to a minimum because excess floating can be detrimental. A light aluminum float carefully and sparingly used will not harm a well finished deck, but will expose poor adjustment and misuse of a good machine. It will also smooth out mortar ridges left by the finishing machine and seal the surface. The Contractor is required to check the deck with a 10-ft straightedge immediately after it is floated.

Low and high spots can possibly be corrected by operating the finishing machine over the area (if the concrete is still plastic).

The Engineer should be cautioned that hard floating of the concrete surface with aluminum floats may cause a chemical reaction between the aluminum and the fresh concrete which could decrease the strength of the concrete at the surface of the concrete. Excessive wear or pitting of the aluminum float could be an indication that chemical reaction is taking place between the float and the concrete.

It is important that the texturing comb be used when the concrete is at the proper consistency. If the concrete is too soft, it will not retain the proper texture obtained by the comb and, if the concrete is too hard, the proper texture will not be achieved. The comb should be set up and ready to use well in advance of the time it will be required. Surface texturing is normally done with a comb except when an overlay is required.

The finished and cured deck slabs must be checked with a 10-ft straightedge and corrected by cutting down the high spots and building up low spots until the entire surface comes within the specified tolerance.

Sidewalks shall be finished smooth with a wood float and then brushed with a fine bristle brush. Use an edger tool at all joints and edges. Block lines on sidewalk surfaces are not desired on structures.

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

Contractors are required to construct a test slab prior to placing bridge deck concrete. The test slab requirement is to ensure that the concrete mix, placement equipment, and workers are all coordinated, operational, and ready to provide a high-quality deck. Contractors may view this work as an unnecessary expense and a source of potential added risk and delay. Contractor RFIs and request to delete the test slabs are common, but WSDOT's experience has been that our deck pours have better outcomes when test slabs are completed. If the Contractor requests to delete the test slab, the Project Engineer has the latitude to delete the test slab(s) when the following conditions are met:

1. The concrete mix has been previously used on a WSDOT project that completed a test slab.
2. The Contractor and key personnel have recently constructed a bridge deck on a WSDOT project.
3. There are no differences in mix design, equipment, or labor from the test slab.
4. The Contractor will not request an increase in the maximum time to discharge.
5. The ASCE and Bridge Office concur with the deletion.

Deletion of the test slab requires a change order and negotiated equitable adjustment credit to the Agency.

SS 6-02.3(10)D3 Concrete Placement

During concrete bridge deck placement, it is important that the amount of concrete placed in front of the finishing machine be kept to a minimum, so it is placed, consolidated, and struck off before it starts to set. Set time may vary depending on a number of factors. The *Standard Specifications* specify that the rate of placement is such that the concrete is placed, consolidated, and struck off within 30 minutes, unless otherwise accepted by the Engineer at the pre-deck pour meeting. The Contractor should know by the pre-deck pour meeting if they will require more than 30 minutes and may request an extension at that time. One example of when they may need to place more concrete in front of the finishing machine is for bridges with extreme skews where concrete is placed to preload girders and equalize girder deflections. The timing should still be as minimal as possible.

SS 6-02.3(11) Curing Concrete

Proper curing of concrete is important to securing strong, good wearing concrete and in reducing cracking. Curing periods and methods specified should be strictly observed.

The last step in ensuring a good concrete job is to provide proper curing. Concrete begins to cure from the time cement and water are added in the mixing chamber and continues for many years after. Concrete is very susceptible to damage during initial curing, if proper steps are not taken. Three of the most important factors are:

1. Surface drying (evaporation).
2. Rapid temperature changes between segments of the concrete as it is curing.
3. Stresses or loads applied before the concrete has reached adequate strength.

All of the specifications regarding curing, form removal, hot and cold weather concreting, etc., are designed to provide protection for the concrete during this critical stage. For example: If the surface begins to dry, the surface will begin to shrink and cracking can occur. To prevent this, the Inspector should be aware that fog misting, curing compounds, wet blankets, plastic sheeting, etc., are designed to be applied before surface drying begins to prevent loss of surface moisture. Some concrete mixes such as microsilica and latex are very susceptible to surface drying and require closer attention due to the effects of thin lift application.

Note: Curing compounds are not chemicals that cure concrete. They prevent water loss by forming a waterproof membrane.

Two Classes (A and B) and Types (1 and 2) of curing compounds are used depending on what is being cured. Class A is a wax resin type of curing compound which can hamper bonding of HMA and pigmented sealer and can cause concrete surfaces to be slick; it is therefore not allowed on travelled surfaces such as bridge decks, bridge approach slabs, and sidewalks. Type 2 curing compound is generally desired because it is white and will reflect solar heat, and it is easier to verify that application quantity is sufficient. Type 1 (clear) is specified when aesthetics are of concern or removal isn't required.

For bridge decks, it is extremely important to keep the finished surface fogged until presoaked burlap can be applied. Also, the burlap should continue to be fogged until soaker hoses and white, reflective sheeting is placed. The presoaked burlap should be applied within one hour after the finishing machine has passed, unless otherwise accepted in the cold weather protection plan or by the Engineer during deck casting. Cold weather and mix design constituents can slow concrete set time, and placing burlap onto concrete that has not attained initial set can damage the deck surface.

Like most materials, concrete expands when heated and contracts when cooled. Therefore, the concrete should not be subjected to extreme temperature changes as hardening takes place.

Hardening of concrete is also slowed down by cooler weather. Concrete must not be exposed to freezing conditions to avoid permanent damage.

Concrete (as it hardens) contains a high percentage of moisture and could crack if the water in the mix freezes and expands. Air entrainment will not protect the concrete from damage during the initial curing period.

Summary

1. Prevent surface moisture loss.
2. Maintain constant temperature (no freezing).
3. Prevent stress loads.

SS 6-02.3(12) Construction Joints

The specifications require that construction joints shall be located and constructed as shown in the plans. Approval to add, move, or delete construction joints must be obtained from the State Construction Office. *Standard Specifications* Section 6-02.3(12) requires that shear keys shall be provided at all construction joints unless a roughened surface is shown in the plans, and where the size of keys is not shown in the plans, they shall be approximately one third of the area of the joint and approximately 1½ inches deep.

Construction joints are to be either vertical or horizontal. Wire mesh, wire lath, and other similar items can be used for a roughened surface construction joint but shall be removed and the joint cleaned before making the adjacent pour. Construction joints in roadway slabs and approach slabs must be formed vertical and in true alignment. An edger shall not be used on the joint but lips and edgings must be removed before making the adjacent pour. If the joint is properly formed, a good straight edge will be obtained with a minimum amount of lips and edgings to be removed.

Shear keys in construction joints shall be formed with 1½ inch thick lumber and shall be constructed the full size shown in the plans. For box girder webs, these shear keys are normally shown in the plans to be full width between stirrups. The specifications require shear key forms to be left in place at least 12 hours after the concrete has been placed. The plans will indicate certain joints to have a roughened surface. These joints shall be finished and prepared for the next pour in accordance with the instructions given in the specifications or as shown in the plans.

Expansion dams or the expansion dam blockout shall be carefully placed before concreting the roadway decks. They shall also be carefully aligned for crown and grade.

Blockouts for expansion joint seals must be carefully formed to the dimensions shown in the plans for proper placement and operation. Be sure to check that the rebar in the blockout does not conflict with the expansion joint anchors. The joint seal must be placed using a lubricant adhesive.

SS 6-02.3(13) Expansion Joints

Bridge expansion joints are installed to accommodate bridge movements while preventing water, salt, and debris infiltration to substructure elements below, thus they must be installed watertight. The *Standard Specifications* require strip seal and compression seal systems to be tested for watertightness by providing a 3 inch minimum head of water for at least one hour. In practice, this is often accomplished by building a trough with plastic sheeting and sandbags and applying a stream of water sufficient to maintain the required water head. Roadway cross-slopes often make it impractical to test the entire joint at once. This can be remedied by performing the test in sections along the joint.

During the test, the expansion joint should be observed from the underside for any signs of leakage. In the case of joints behind abutments without underside access, the joint should be observed from the sides and front face of the abutment. Any amount of water observed is cause for repair.

SS 6-02.3(14) Finishing Concrete Surfaces

As soon as possible after the forms are stripped, the concrete surfaces shall be examined and all lips or edgings where form boards have met, shall be removed with a stone or sharp tool. Bolt holes and rock pockets shall be filled with cement mortar and floated to a smooth finish. The mortar patch shall be the same color as the adjoining concrete surfaces. Finishing of concrete surfaces shall be done in accordance with the provisions of the *Standard Specifications* and special provisions.

The amount of work necessary to complete the finishing satisfactorily, depends entirely on the quality of the original concrete work. If the forms have been poorly constructed and the concrete surfaces are rough and uneven, it will be necessary for the Contractor to do sufficient rubbing and finishing after the forms are removed to secure a satisfactory job. Grinding leaves a surface that is off color and should be kept to a minimum.

The primary purposes of finishing formed surfaces are:

- To seal the surface from water and other elements that can rust or corrode metal ties and reinforcement within the concrete.
- To provide a uniform, pleasing appearance for surfaces that will remain visible to the public.

SS 6-02.3(14)A Class 1 Surface Finish

- All rail bases, curbs, traffic barriers, pedestrian barriers, and ornamental concrete members.
- As designated in the Plans and in accordance with *Standard Specifications* Section 6-02.3(14).

SS 6-02.3(14)B Class 2 Surface Finish

- Required for all other surfaces.

See the *Standard Specifications* for additional requirements.

SS 6-02.3(17) Forms and Falsework

Falsework construction is a critical part of the bridge construction process. Generally, the factor of safety used for design of falsework is less than that of permanent construction. Therefore, it is extremely important that the falsework is constructed in accordance with the falsework drawings. Any changes to the falsework drawings must be reviewed by the Bridge and Structures Office.

The forms for the structure shall be constructed in accordance with the falsework and form plans and the requirements of *Standard Specifications* Section 6-02.3(17). In general, the forms used for all concrete surfaces which will be exposed, shall be faced with plywood. All plywood used shall be exterior type except where CDX is allowed by the specifications. All forms have to be strong enough to hold the plastic concrete in place until it has hardened. Forms should be designed to permit easy removal without damage to the concrete. Forms are a critical part of the concrete bridge construction process. Generally, the factor of safety used for design of forms is less than that of permanent construction. Therefore, it is extremely important that the forms are constructed in accordance with the form drawings. Any changes to the form drawings shall be reviewed by the State Bridge and Structures Office.

The Contractor is responsible for designing and constructing the forms and falsework for fixed-form concrete. The Contractor must submit detailed plans and calculations in accordance with Section 6-02.3(16):

Prior to placing concrete, the Inspector should verify that all forms:

- Provide forming faces that are:
  - Smooth and firm.
  - Clean of dirt, laitance, oil, or any other material that would contaminate or discolor the concrete.
  - Treated with an approved form-release agent.
- Are mortar tight to avoid any leakage (including tape or caulking if needed for surfaces that will require Class 2 finish).
- Are constructed in accordance with the forming plans.
- Are adequately rigid and well supported to hold and retain the concrete without distortion or displacement.
- Are set at the locations, dimensions, lines, and grades as specified in the plans.

If wood forms are used, see that plywood is used for the form faces with:

- The joints and grain generally in line with the line of the structure.
- The face grain of the plywood running perpendicular to the supports.
- No offsets or projections that would leave an impression in the concrete surface.

Also verify that:

- Uniform chamfer strips are set at the correct line and grade as required for filleted edges.
- Adequate tie rods, snap-ties, hairpins, studs, walers, and braces are securely placed as needed support.

If metal or fiberglass forms are used, the same basic requirements apply, but particularly check for:

- Any dents or other defects that would harm the uniformity of the concrete surface.
- Any rust or other foreign material that would discolor the concrete surface.
- Countersunk bolts and rivet heads.
- Adequate support clamps, rods, and pins.

Prior to placing any reinforcing or concrete loads on the falsework, verify that:

- The bottom of the falsework is set on a solid foundation, with mudsills, minimum pile diameter, etc., all constructed per plans.
- The upper portion provides firm, uniform support.
- Devices such as screw-jacks and wedges are used to hold the forms at the correct elevation, and that they are free from defects, and undamaged or not bent.
- When wedges are used, they are placed in pairs to provide uniform bearing.
- The falsework construction is in accordance with the falsework plans and the *Standard Specifications*.

Major failures with loss of life have occurred as a result of poor falsework and formwork construction. It is critical that the Inspector check these temporary structural elements very carefully. Any deficiencies must be corrected before construction loads are applied. If there is a question, the State Bridge and Structures, Construction Support Engineer, or the State Construction Office should be contacted.

Suggested acceptance tolerances are as follows:

1. Bridges and similar structures:

   a. Variation from the plumb or the specified batter in the lines and surfaces of columns, piers, walls, and abutments.

Exposed, in 10 feet	½ inch
Backfilled, in 10 feet	1 inch

   b. Variation from the level or from the grades indicated on the drawings in slabs, beams, horizontal grooves, and railing offsets.

Exposed, in 10 feet	½ inch
Backfilled, in 10 feet	1 inch

   c. Variation in cross-sectional dimensions of columns, piers, slabs, walls, beams, and similar parts.

Minus	¼ inch
Plus	½ inch

   d. Variation in thickness of bridge slabs.

Minus	⅛ inch
Plus	¼ inch

   e. Footings: Variation in dimensions in plan.

Minus	½ inch
Plus	2 inches

   f. Misplacement or eccentricity 2 percent of the footing width in the direction of misplacement but not more than 2 inches.

   g. Reduction in thickness.

Minus	5 percent of specified thickness

   h. Variation in the sizes and locations of slab and wall openings ½ inch

Forms for concrete surfaces which will be exposed shall be treated with a parting compound consisting of a chemical release agent. Form oil or other oils shall not be used. The parting compound shall be applied before the reinforcing steel is placed. The forms shall be thoroughly wetted on both sides in advance of placing the concrete.

The basic requirements for the removal of any forms and falsework are that:

- The curing temperature was above 50°F during the cure period and that strength is adequate.
- No forms or falsework may be removed until the minimum time has been met as listed in Section 6-02.3(17)N or as authorized by the Engineer.
- All forms and falsework must be removed unless there is no access for removal (i.e., inside a box girder bridge).
- All forms and falsework must be removed in a manner that will not damage the structure.

Timing is a key consideration in the removal of forms and falsework. In terms of curing, the concrete, forms, and falsework must remain until the concrete has sufficient strength to support itself. For finishing purposes, it is generally better to remove the forms as early as possible to finish the surface while it is still green. Therefore, the timing of falsework and form removal depends largely on the type of structure as well as how it is cured and finished. If forms are removed during the required curing period, the Contractor shall provide the required curing method to the exposed concrete surface as described in Section 6-02.3(11).

SS 6-02.3(24) Reinforcement

For most concrete structures, some type of reinforcement is required to resist high tension stresses. Reinforcing materials include:

- Uncoated deformed steel bars, which are most commonly used.
- Other types, such as welded wire reinforcement epoxy-coated bars, wire, prestressing cable.

  Note: Epoxy-coated bars require special handling to prevent damage to the coating.
- Wire ties and other devices to securely hold the reinforcement in place.

The Contractor is responsible for determining and ordering quantities from the plans.

As reinforcing steel is delivered and stored at the project site, the Inspector should verify that:

- All positioning, spacing, sizes, lengths, shapes, and splice locations conform with the plans.
- Any field bending is done as specified and any cracked or split bars are rejected. If in doubt, reject the bar in question.

The Inspector should verify that the reinforcing placed is:

- Tied at all intersections if bar spacing is 1 ft or more.
- Tied at alternate intersections if spacing is less than 1 ft.
- Supported in accordance with the *Standard Specifications*.
- Tack welding is not allowed. It can severely damage the reinforcing steel.
- Check that clearances between the forms and the reinforcement are within ¼ in of those specified in the plans.
- Check that splices are located and constructed only as shown in the plans using either:
  - Lap splicing:
    - Not permitted for No. 14 or No. 18 bars.
  - Welded splices:
    - Special inspection is required (steel fabrication inspector).
    - Advance review of welding procedures.
    - By certified welders (test welds).
  - Mechanical splicing (if allowed in the plans):
    - This type of splice must be approved by the State Materials Lab before use.
    - Check that reinforcement is securely supported and held in place as follows:

  - By preapproved metal or plastic chairs, hangers, support wires, or mortar blocks that are at least as strong as the structure (mortar blocks require manufacturer certification).
  - With such supports having the correct dimensions to provide the required clearances.
- Check that all damaged epoxy-coated rebar is repaired in accordance with the *Standard Specifications*.

See the Bar Identification Guide (Figure 6-2) for proper identification of rebar at the job site.

The ASTM specifications for billet-steel, rail-steel, axle-steel, and low-alloy steel reinforcing bars (A 615M, a 616M, a 617M, and a 706M respectively) require identification marks to be rolled into the surface of one side of the bar to denote the producer's mill designation, bar size, type of steel and minimum yield designation (see Figure 6-2). Grade 60 bars show these marks in the following order:

1st – Producing Mill (usually a letter)

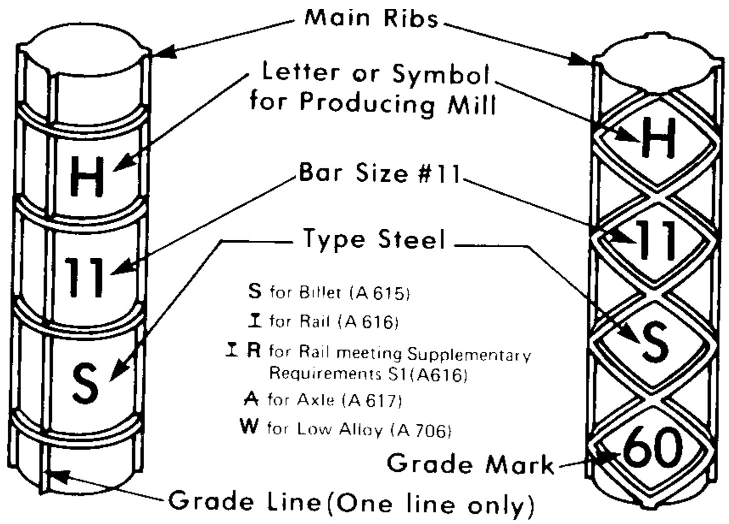
2nd – Bar Size Number (#3 through #18)

3rd – Type Steel:

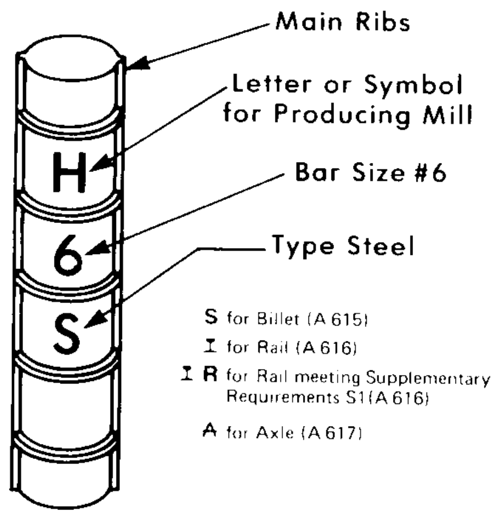
S for Billet meeting Supplemental Requirements S1 (A 615M)
N for New Billet (A 615M)
R for Rail meeting ASTM a 617M, Grade 60 bend test requirement (A 616M) (per ACI 318-83)
I for Rail (A 616M)
A for Axle (A 617M)
W for Low-Alloy (A 706M)

4th – Minimum Yield Designation

Figure 6-2



GRADE 60 (400)



GRADE 40 (300) AND 50 (350)

Minimum yield designation is used for Grade 60 bars only and can either be one (1) single longitudinal line (grade line) or the number 60 (grade mark).

A grade line is smaller and between the two main ribs which are on opposite sides of all U.S. made bars. A grade line must be continued at least 5 deformation spaces. A grade mark is the 4th mark on a bar.

Grade 40 and 50 bars are required to have only the first three identification marks (no minimum yield designation).

Bar identification marks may be oriented as illustrated or rotated 90 degrees. Grade mark numbers may be placed within separate consecutive deformation spaces. Grade line may be placed on the side opposite the bar marks.

Reinforcing steel shall be placed in position as shown on the plans and held securely during the placement of the concrete. The strength of a reinforced concrete structure depends not only upon the amount of steel placed but also on its proper location. Improper location of the steel can impair the strength of the structure.

In instances where reinforcing steel is shown in detail in specific relationship to other material and details such as inserts, openings, etc., the Inspector should make sure that this relationship exists when inspecting the placement of the reinforcing steel. If the shown relationship is impossible to maintain or results in a conflict with other details, the State Construction Office shall be consulted to obtain clarification of the details.

The reinforcing steel shall be securely blocked from the forms by means of small mortar blocks, with a groove or tie wire embedded, not more than 2 inch square, or by other approved devices. If metal chair supports are used as supports for steel reinforcing bars, all surfaces of the chair supports not covered by at least ½ inch of concrete shall be treated in accordance with the requirements of *Standard Specifications* Section 6-02.3(24)C.

Runways for wheelbarrows or concrete buggies used in placing concrete shall not be supported on the steel reinforcing bars.

Steel delivered to the job far in advance of its use should be stored under cover to prevent rust. Mill scale is sometimes present on the reinforcing steel to such an extent that it must be removed. This is especially true with the larger bars. Removal can usually be accomplished by the use of wire brushes or by tapping the bars with hammers. Hardened concrete mortar must be removed from the reinforcing steel before placing the concrete. All reinforcing steel shall be in its proper place before concrete is placed. Driving of dowels, rail bars, etc., into concrete (wet setting) shall not be permitted. See the *Standard Specifications* for further details.

Before concrete is placed, the reinforcing steel shall be inspected to see that it conforms to the plans and that the steel is properly fastened in position. The amount of cover of concrete over the reinforcing steel in bridge roadway slabs and bridge approach slabs is critical. The Inspector must verify compliance with plan dimensions in the slabs by an adequate number of measurements of the steel reinforcing bar locations in the forms before and immediately after placing concrete. These measurements can be taken at the same time checks on the depth of the concrete in the slabs are taken. These measurements shall be recorded as to depth and location and made a part of the project construction documents.

When steel reinforcing bars protruding from columns or walls are exposed to weather for several months, they rust and exposed surfaces below become stained with rust. To prevent this, the bars should be protected to prevent rust. Coatings used for this purpose may prevent adequate bonding of concrete to the steel bars and should be removed from the bars before concrete is placed, except as allowed by the *Standard Specifications*.

SS 6-02.3(24)E Welding Reinforcing Steel

Reinforcing bars shall not be welded unless welding is indicated in the plans or special provisions. If welding is specified, the WSDOT welding inspector must be contacted for purposes of certifying welders and procedures. Reinforcing bars which are to be welded must be furnished of steel which is suitable for welding as specified.

Only operators qualified as specified in *Standard Specifications* Section 6-02.3(24)E shall be allowed to weld reinforcing steel.

AWS specifications require that Low Hydrogen type electrode (welding rod) be used for welding reinforcing steel. Generally, grade E7018 electrodes shall be used for grade 40 reinforcing bars and grade E8018 electrodes shall be used for grade 60 reinforcing bars. If semiautomatic welders are used equivalent grade electrodes shall be used. It is important that moisture be eliminated from the electrode and the steel reinforcing bars. The electrode must be prepared as called for in *Standard Specifications* Section 6-03.3(25). To do this, a drying oven is essential and must be available and used at the site where welding is done.

The recommended procedure for welding steel reinforcing bars is given in *Standard Specifications* Section 6-02.3(24)E. The Contractor shall submit a welding procedure to the Engineer for review. The Project Engineer shall transmit the Contractor's welding procedure to the State Bridge and Structures, Construction Support Engineer for review.

SS 6-02.3(25) Prestressed Concrete Girders

Shop inspection of the manufacturing process of prestressed concrete products will be done by an inspector working under the direction of the State Materials Engineer. The State Materials Laboratory has instituted a procedure of inspecting each prestressed concrete plant in the State on an annual basis. During this inspection, the State Materials Laboratory obtains a list of the sources of the component parts to be used in manufacture of the prestressed concrete members. When the Contractor submits a request for approval of source of prestressed products, the complete member and the prestress plant which will manufacture it need only be listed.

The Inspector prepares a weekly Fabrication Progress Report and Inspectors Daily Report, and submits them to the Project Engineer for information and records. When the prestressed unit is completed, including finishing, the Inspector will attach an Approved for Shipment tag, and/or the girder will be stamped with an "approved for shipment" and a lab I.D. number. The Approved for Shipment tag properly signed and dated or the "approved for shipment" and a lab I.D. number will be the Project Engineer's basis for accepting the product at the job site. The Project Engineer will be required to inspect the item only for any damage which may occur during shipment or after the item arrives at the job site.

Finishing of concrete surfaces of prestressed units shall be in accordance with *Standard Specifications* Sections 6-02.3(14) and 6-02.3(25)H unless specifically changed by the special provisions. The Shop Inspector shall require that the finishing done in the shop is in accordance with the specifications.

Prestressed concrete girders shall be maintained in a plumb, upright position at all times and shall be lifted by means of the lifting strands provided at the ends of the girders. All prestressed girders have been designed for a vertical pickup at the ends as indicated in the contract plans, and any other method will induce stresses which could cause failure of the girder during pickup. Some deviation from the vertical is safe for some girders. If the Contractor wishes to deviate from the vertical pickup, they shall have the proposed method analyzed by their engineers and shall submit the method, with supporting calculations, for review. The Project Engineer submits the calculations to the State Construction Office for review. If the girders are broken or damaged during handling or erection, they will have to be replaced at the Contractor's expense.

The girders shall not be placed on the finished piers or abutments until the concrete in the piers or abutments has obtained at least 80 percent of its design strength. If grout pads are required, they shall be constructed and cured as required by the plans and specifications before placing the girders. Elastomeric bearing pads conforming to Section 9-31.8(1) shall adhere to the concrete surface using the manufacture's recommended adhesive product prior to placing the girders. The girders must meet the dimensional tolerances listed in *Standard Specifications* Section 6-02.3(25)I.

SS 6-02.3(25)A Shop Drawings

The Contractor is required to submit shop detail plans to the Project Engineer for review. The Project Engineer shall check these plans for compliance with the contract plans and specifications.

Manufacture of these members shall not begin until the Contractor has received comments on the method, materials, and equipment they propose to use in the prestressing operations. Deviations from the shop drawings shall not be permitted.

Welding of the reinforcing bars will not be permitted unless shown in the contract plans.

The State Materials Lab has published a manual entitled “Inspectors Guide for Prestressed Plant Inspection and Quality Control” which contains more detailed instructions for this work.

SS 6-02.3(25)K Vertical Deflection

Precast prestressed girders start creeping up immediately after prestressing strands are released in the casting bed. Over time, creeping or girder deflection upward continues. Bridge plans estimate the expected creep at 120 days, from prestress release to deck placement, and designate the letter “D” for this deflection. Theoretical girder camber at mid span vs. Actual girder camber measured in field, after girder erection, should be compared for compliance with *Standard Specifications* Section 6-02.3(25)K.

The camber diagram is a parabolic curve. In order to have a smooth vertical profile the pad dimension on top of girder flange varies through the length of span (see Figure 6-3). This dimension is usually least (depending on the vertical profile curve) at center span and maximum at center line of bearings which bridge plans refer to as “A” dimension. The designation “C” is the amount of camber added to the deck grade elevations to account for the anticipated downward girder deflection due to all superimposed loads (slab, overlay, sidewalks, utilities and traffic barriers).

Finished roadway grade elevations should be calculated at tenth points along the centerline of each girder web and from centerline of bearing to centerline of bearing. The elevations for girders exceeding 100 feet in length shall be computed at equivalent intervals not to exceed 10 feet. Camber values at these locations need to be added to the finished roadway grade elevations to compensate for the girder deflection due to superimposed loads. Equation 6-1 calculates the camber at any point along the span.

$$Y = C - 4C\,(M - 0.5)^2 \qquad \text{(Equation 6-1)}$$

Where

Y	=	camber at any point along the span length in inches
C	=	deflection due to superimposed dead load at span mid point in inches
M	=	location of span in decimal percent

The following example shows how tenth point span camber can be calculated.

Example:

Calculate camber at 0.20-point span for a prestress girder when girder length (ctr. - ctr. bearing) is 174.2 feet and “C” dimension at mid span given as 3 inches (see Figure 6-4).

$$Y = 3 - 4(3)(0.20 - 0.5)^2$$
$$Y = 1.92 \text{ inches}$$

Once the girders are set in place and before any load is added to the girders, elevations are taken at the tenth point locations (the elevations for girders exceeding 100 feet in length shall be computed at maximum intervals of 10 feet) to be used to determine an adjusted “A” dimension. The adjusted “A” dimension is determined by subtracting the as built elevations from the calculated finished roadway grade elevations plus camber to determine the new adjusted “A” dimension at each location. The adjusted “A” dimension is used to string line between two adjacent points to determine soffit location.

Figure 6-3

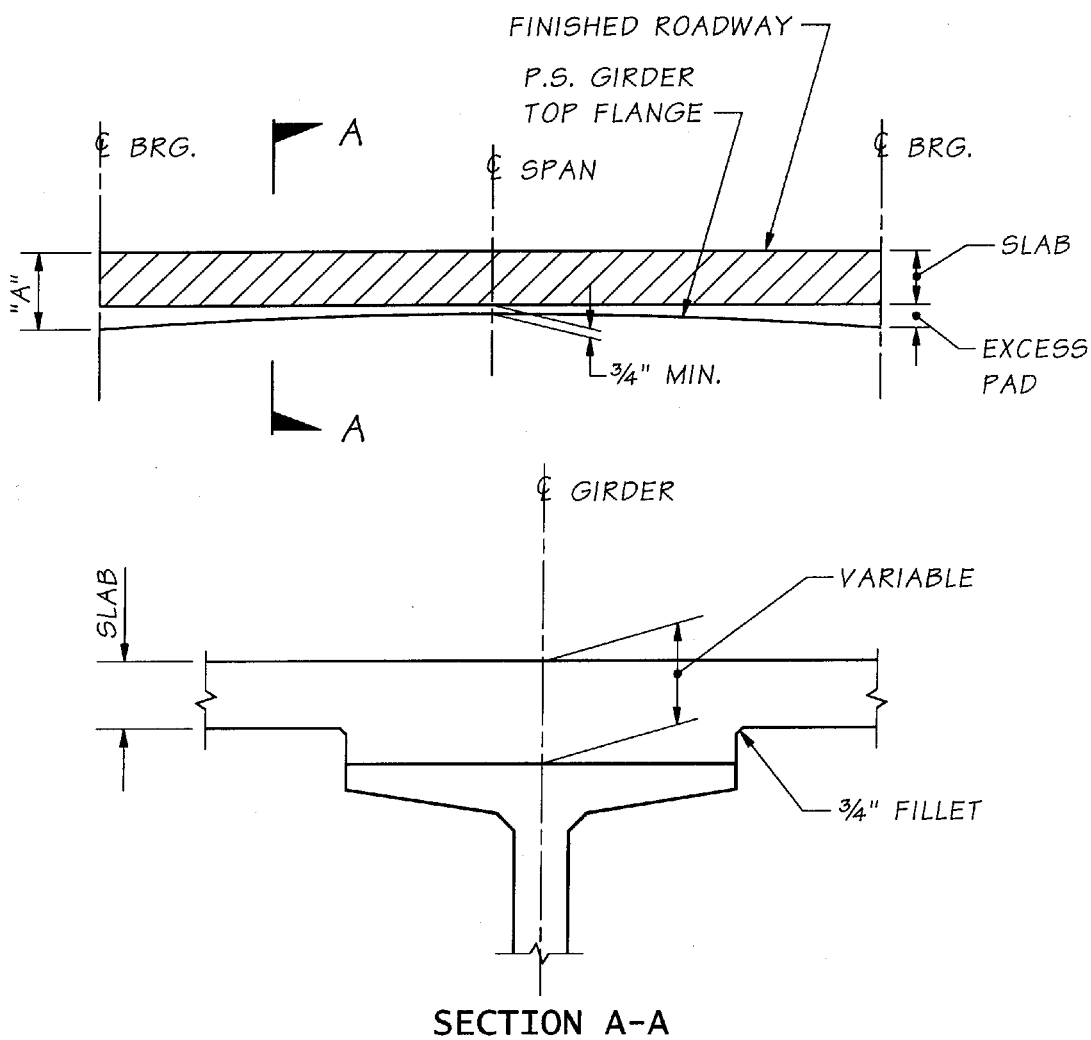
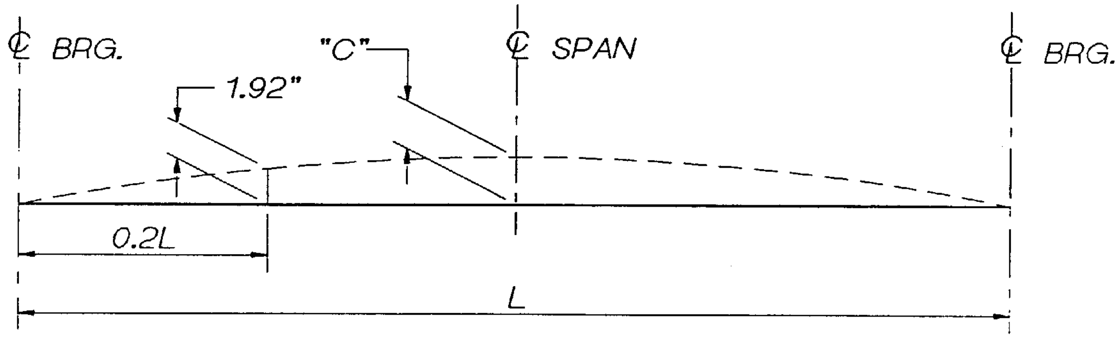


Figure 6-4



SS 6-02.3(26) Post-Tensioned Concrete

The construction of cast-in-place post-tensioned bridges requires considerable attention to details of construction by the Contractor and Inspectors. The State Construction Office is available to present job-specific training on post-tensioned bridges. They should be contacted after the post-tensioning shop drawings have been reviewed and before post-tensioning ducts and anchors are to be placed.

In addition to the falsework and form plans for the structure being reviewed by the Bridge and Structures Engineer, post-tension detail plans shall be submitted for review as shown in the Shop Plans and Working Drawings Table in Section SS 1-05.3 and Figure 1-1. Included in these details will be the anchoring details, jacking forces, lift off forces, tendon profile, elongation of the tendons, and the tendon stressing sequence. In many structures, the dead load of the structure is increased at the jacking ends during the jacking operation. In these cases, the falsework at the jacking ends must be designed to carry the additional dead load.

The installation of the post-tension system begins with the placing of assemblies consisting of bearing plate, transition cone or trumpet and grout inlet. Duct sections consisting of rigid conduit are assembled with couplers and are tied to the stirrups. Anchorages and bearing plates are securely fastened to the forms to prevent movement and loss of mortar during concreting. Connections between trumpets and ducts, ducts and couplers, and ducts and vent saddles are taped with a durable and waterproof tape to prevent intrusion of mortar.

It is necessary that the ducts be located in the position shown in the post-tension details in order for the structure to function as designed. A misaligned duct will cause increased friction and localized stress which can result in failure of the member during the stressing operation. The Inspector must check to see that the ducts are properly located and securely fastened in place to prevent movement during concreting.

On continuous structures, vents must be placed at the high and low points of the tendon and grout inlets at the ends of the tendon.

At the completion of the duct installation and prior to placement of concrete in the top slab, a device of slightly smaller diameter than the inside diameter of the duct shall be blown through the ducts to ensure no undetected damage or blockage has occurred (see *Standard Specifications* Section 6-02.3(26)E.

The prestressing reinforcement strand is delivered to the site in sealed reel-less packs or reels containing desiccant to prevent corrosion. It is necessary that the prestressing reinforcement is free of rust and kept clean while it is assembled, stressed, and grouted. Normally, the grouting shall take place within 10 days of the time the strand is removed from the packs to prevent the accumulation of rust. The Inspector should check the reels of strand intended for use and reject those which show damage to the strand or visible rust. See *Standard Specifications* Section 6-02.3(26)F for further requirements.

Some projects may be designed for the use of high strength steel rods instead of the strand. These rods come in various sizes to give the required steel area for the tendon in one bar instead of bundling several strands in the tendon.

Jacking operations shall not be started until the concrete in the structure has cured for the specified time or reached the specified strength. Jacking shall be carried out in the sequence shown on the post tension details to minimize the amount of eccentric loading on the structure. **During the jacking operations, no person should be directly behind either end of the tendon.** Occasionally a tendon will let go, resulting in a very dangerous situation.

Each jack used to stress tendons shall be equipped with either a pressure gauge or a load cell along with certified calibration charts for determining the jacking force.

Gauging devices should be re-calibrated at intervals of not more than 180 days; however, if during the progress of the work, any gauging system appears to be giving erratic results, or if gauge readings and elongation measurements indicate materially different stresses, the jack and the gauges shall be re-calibrated.

A starting load, usually 20 percent of the jacking load, as shown in the post tensioning schedule, is applied to the tendon. The purpose of this starting load is to take up the slack in the tendon so that an accurate elongation measurement may be made. This load is applied by hydraulic jacks and measured by the jack gauges. During the stressing operation, the tendons shall be jacked to the specified load and the jacking load and elongation shall be recorded. Also the elongation after seating must be measured and recorded (see Figure 6-5).

In the event of discrepancies between measured elongations and calculated elongations (see Stress Acceptance Criteria), the entire operation should be carefully checked and the source of error determined and corrected before proceeding further. A discrepancy between the elongation and the jacking force usually indicates that the gauge on the jack is not correctly calibrated, there is undue friction between the duct and the tendon, or the tendons are not properly anchored.

Stress Acceptance Criteria

Strand Tendon (lengths 50 feet and less):

1. The tendon may be accepted provided: The measured elongation is equal to or exceeds 93 percent of the calculated elongation.
2. A force verification lift-off is performed: The verification lift-off force is between -5 percent and +5 percent of the calculated force.

Strand Tendon (lengths greater than 50 feet **and less than 150 f**eet**):**

1. If the measured elongation is between -7 percent and +7 percent of the calculated elongation, the tendon can be accepted.
2. If the measured elongation exceeds 107 percent of the calculated elongation, confirm the jack/gauge calibration, and then perform a force verification lift-off:
   a. If a force verification lift-off is performed on one end of the tendon only and the lift-off force is between -1 percent and +5 percent of the calculated force, the tendon can be accepted.
   b. If a force verification lift-off is performed on both ends of the tendon (jacking end and anchor end) and the lift-off forces are between -5 percent and +5 percent of the calculated force, the tendon can be accepted.

Strand Tendon (lengths 150 feet **and greater):**

1. If the measured elongation is between -7 percent and +7 percent of the calculated elongation, the tendon can be accepted.
2. If the measured elongation exceeds 107 percent of the calculated elongation, confirm the jack/gauge calibration, and then perform a force verification lift-off.
   a. If a force verification lift-off is performed on one end of the tendon only and the lift-off force is not less than 99 percent of the calculated force nor more than $0.7\ f'_s\ A_s$, the tendon can be accepted.
   b. If a force verification lift-off is performed on both ends of the tendon (jacking end and anchor end) and the lift-off forces are not less than 95 percent of the calculated force nor more than $0.7\ f'_s\ A_s$, the tendon can be accepted.

Singularly Jacked Four-Strand Transverse Deck Tendon:

The tendon may be accepted provided:

1. The measured elongation of an individual strand is between -10 percent and +10 percent of the calculated elongations.
2. The average of all four individual strand percent elongations is between -7 percent and +7 percent of the calculated elongation.

Bar Tendon:

1. The tendon may be accepted provided: The measured elongation is equal to or exceeds 93 percent of the calculated elongation, and
2. Perform a force verification lift-off: The verification lift-off force is between -5 percent and +5 percent of the calculated force.

If acceptance tolerances are exceeded, notify the State Construction Office.

f'_s = specified minimum ultimate tensile strength of prestressing steel (270 ksi for strands and 150 ksi for bars.)

A_s = cross-Section area of the tendon (0.153 square inch for ½-inch diameter strand, 0.217 square inch for 0.6 inch diameter strand.)

The grout used is fluid and quite different from the mortar we usually associate with the term grout. The component materials of the grout mix must be accurately measured. The maximum amount of water specified must not be exceeded. The grout should be screened after it has been mixed and before it is added to the grout equipment to remove lumps which might cause clogging of the ducts.

Immediately, prior to grouting, the ducts shall be blown out with oil free compressed air. Grout is applied continuously by pumping under moderate pressure at the lower end of the duct toward an open vent at the upper end until all entrapped air is forced out the open vents. The open vents are closed under pressure of issuing grout after a steady solid stream of grout is discharging. The grouting pressure is gradually increased to a minimum of 100 psi and 200 psi maximum and held at this pressure for a minimum of 10 seconds. The grouting entrance is then closed.

After grouting of the tendons, the recesses for the anchorages are cast solid with concrete.

A complete record must be kept of the stressing operations.

An example of the Post-Tensioning Record (DOT Form 450-005) is shown in Figure 6-5. The following explanations will help in completing the record:

A. Required jacking force for the tendon is obtained from the post-tensioning details.

B. Gauge pressure is obtained from the certified calibration chart for the jack to obtain the required jacking force listed in “A” above.

C. Gauge pressure for the initial force to take up the slack in the tendon and is usually 20 percent of the force obtained in “B” above.

D. The designed elongation is obtained from the post-tensioning details, however the stress strain curves prepared by the steel manufacturer shall be used to determine the modulus of elasticity for adjusting the designed elongation based on the average value of all strands to be incorporated in the tendon.

E. This required seating take up is obtained from the post-tensioning details. This is usually ¼ inch to ⅜ inch.

F. & G.
The elongation must be measured at the initial force of 20 percent of the required jacking force, at the specified jacking force, and again at the 20 percent loading.

H. The difference in the elongation measured at full force and the elongation measured at the initial force of 20 percent (minus any dead end slip). This elongation should be reasonably close (see Stress Acceptance Criteria) to the required elongation in “D” above.

I. Seating take-up is the difference in the elongation measured at full force and the elongation measured after the tendon has been seated and the jacking force reduced to the initial force of 20 percent of full force. However, since the elongations are measured at the end of the jack, the elongation of the tendon from the wedges to the measuring point must be accounted for to obtain the true seating takeup. After finding the difference between the full jacking force elongation and the 20 percent of full jacking force, (I1) the elongation of the tendon inside the jack must be subtracted from the difference to obtain the true seating take-up. (I2) The elongation of the tendon inside the jack is approximately 1⁄16 inch per foot. This seating take-up should be the same as the required take-up in "E" above. It is important that the specified seating take-up be obtained as it has an appreciable effect on the stress in the tendon.

J. Percent elongation per tendon is a comparison of the calculated elongation and the measured elongation. If the elongation obtained at full jacking force is not reasonably close to the required elongation, the following conditions are usually indicated:

- There is more (or less) friction in the tendon than was anticipated in the calculations of the post-tension details.
- The gauging devices on the jack are not properly calibrated.
- The strands of a tendon are not properly anchored.

If tendon stressing is performed at an air temperature below 60°F, the Contractor should not be allowed to use jack pressure gauges that utilize oil or glycerin. This will ensure accurate jack pressure readings. The reason for this is that these gauges tend to react slowly at lower temperatures. What can happen with these gauges is the jack operator will bring jack up to the required gauge pressure and shut the jack off. Since the gauge is slow in reacting, it will continue to rise until it "catches" up, resulting in over stressing the tendon. Once this occurs, the tendon will usually need to be replaced.

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- The gauging devices on the jack are not properly calibrated.
- The strands of a tendon are not properly anchored.

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Figure 6-5 Post-Tensioning Record (DOT Form 450-005)

Washington State Department of Transportation

Post-Tensioning Record

Contract No.	Page No
Section	
Structure	
Project Engineer	Inspector

Date	Gir. No.	Tendon No.	Jack No.	Jack Location	Req'd. Jacking Force Per Tendon (Kips)	Strands Per Tendon	Gauge @ 20% Jacking Force (p.s.i.)	Gauge @ Req'd. Jacking Force (p.s.i.)	Gauge @ Actual Jacking Force (p.s.i.)	Actual Elong (in.)	(A) 100% Actual Elong. - 20% Actual Elong. (in.)	(B) Calc. 80% Elong. (in.)	% Elong. Per Tendon	100% Actual Elong. - Seated Elong. (in.) = (c)	(c) - Jack Elong. = Seating Take-up (in.)	Req'd. Seating Take-up (in.)
1-2-96	A	1	6	PIER 1	372	12	1400	7000	1400	4 1/4						
									7000	23 1/2	19 1/4	19 1/8				
									1400	22 7/8				5/8	7/16	3/8
							DEAD	END	SLIP		(-) 1/8					
											19 1/8			5/8 - 3/16		
														3 FOOT	JACK X	1/16 IN/FT
1-2-96	A	1	6	PIER 3	372	12	1400	7000	1400	∅						
									7000	1 3/4	1 3/4	2				
									1400	1 1/4				1/2	5/16	3/8
											20 7/8	21 1/8	99			
					A		C	B	F	G	H	D	J	I1	I2	E

Distribution: Final Records
State Construction Office

Note: %Elong. = The sum of columns "A" for both ends of the tendon divided by the sum of columns "B" for both ends of the tendon X 100.
% Elong. shall be between 93% minimum and 107% maximum.

DOT Form 450-005 EF
Revised 3/2002

6-03 Steel Structures

SS 6-03.3(7) Shop Plans

The Contractor shall submit shop plans of all steel fabrication for review. Fabrication of the steel shall not be started until the shop plans have been reviewed by the Bridge and Structures Engineer (or Terminal Design Engineer for the Ferries Division projects) and the materials source and fabricator have been given approval by the State Materials Engineer. The State Materials Engineer shall advise the State Bridge and Structures Engineer (or Terminal Design Engineer) when the materials source or fabricator has been approved. The plans will not be returned to either the Contractor or the fabricator by the Project Engineer until the approval of source has been given by the State Materials Engineer. WSDOT reviews the shop plans for sufficiency of the materials and connections and not for the correctness of dimensions. Some details of the design drawings may, with the approval of the State Bridge and Structures Engineer (or Terminal Design Engineer), be changed to suit the erection methods the Contractor desires to use. These revisions may require a change order.

The Contractor shall submit eight sets of all shop detail plans required for fabrication of the steel directly to the State Bridge and Structures Engineer and two sets to the Project Engineer. For the Ferries Division projects, all ten sets shall be submitted to the Terminal Design Engineer. If a railroad is involved, four additional sets are required for each railroad involved. See the shop plans and working drawings table in Section SS 1-05.3 and Figure 1-1. The Project Engineer should advise the State Bridge and Structures Engineer of any conditions that would affect the checking and review of the drawings. These comments should be shown with a green color marker on the Project Engineer's copy.

Shop inspection is performed either by inspectors or representatives of the State Materials Laboratory. Material Acceptance Reports are obtained by these inspectors and provided to the Project Engineer upon completion of the shop fabrication. Erection plan sheets generally accompany the shop plans.

Prior to completion of the project, the Contractor is required to furnish shop drawings on mylar or equivalent, which will be sent to the State Bridge and Structures Office for their permanent file. These drawings must be suitable for reproducing by microfilming.

SS 6-03.3(7)A Erection Methods

Falsework and erection plans for structural steel structures shall be submitted for review in the same manner as for concrete structures.

Camber diagrams are normally shown in the contract plans. It is the Fabricator's responsibility to fabricate the members to the prescribed camber shown in the plans. The Fabrication Inspector should verify that the members are fabricated in accordance with the shop drawings.

The use of heavy equipment for erection purposes requires the review of the State Bridge and Structures Engineer. See *Standard Specifications* Section 6-01.6.

Laying out work for structural steel spans requires greater accuracy than for other structures. Use precise instruments, standardized tapes, scales and thermometer when making layout. Spacing of piers, bents, and anchor bolts shall be as shown in the plans, providing the span after fabrication in the shop is the correct length.

The fabrication shop is required to furnish a sketch showing the length of span and amounts of camber measured in the shop at the time the spans are assembled. The Project Engineer should have a copy of this sketch before erection is begun. The lengths as measured in the shop seldom vary more than ¼ in to ⅜ in from the design drawings, and there is sufficient play in the anchor bolt sleeves for this tolerance.

Allowance will be made on the design drawings for stretch of the span due to loss of camber. The Project Engineer shall compute camber elevations from the shop camber measurements taken by the shop. Elevations shall be set above the falsework at each panel point for the camber blocking. Most erectors set the camber blocks high to allow for settlement of the falsework. The amount of allowance for settlement should be decided by the erector. The Project Engineer shall give the exact elevations for the finished camber. Elevations shall be given and carefully checked as an error means that an unnecessary amount of jacking and adjusting may be required.

The adjustment of spans is often a source of argument between erectors and engineers. Accurate work on the part of the Engineer will do much to avoid such arguments. Elevations set on the falsework before the load is applied may not be correct after the load is applied. It is the responsibility of the Contractor to determine the allowance that may be necessary to compensate for settlement in the falsework. It is easier to lower the span than to raise it.

SS 6-03.3(9) Handling, Storing and Shipping of Materials

Structural steel members shall be handled carefully to prevent twisting, bending, or scraping the member. The material shall be supported on suitable skids or platforms to keep it off the ground or out of water and it shall be protected from deterioration by rust.

Structural steel members should not be unloaded and stored on adjoining concrete approach spans. If the Contractor proposes to use the concrete approach spans to support the structural steel members, the proposal must be submitted in writing to the Bridge and Structures Office for review. This proposal shall include drawings describing the support locations, loads, and supporting stress calculations. The structural steel members shall be placed on timber blocking, spaced so that the weight will be carried on the girders (load carrying members) and not on the comparatively thin concrete deck slab. Bridge decks are designed for carrying traffic and not as storage or dock space. This is especially true for concrete sidewalk slabs. Sidewalk concrete slabs shall not be overloaded by loads such as building material, tool sheds, or paint sheds.

SS 6-03.3(10) Straightening Bent Material

Methods for straightening of plates, angles, other shapes, and built-up members shall not produce fracture or other injury to the metal, and shall be reviewed by the State Construction Office. Distorted members shall be straightened by mechanical means or by the carefully planned and supervised application of a limited amount of localized heat. The temperature of the heated area shall not exceed 1,100°F (a dull red) and shall be controlled by temperature indicating crayons, liquids or bimetal thermometers.

Following the straightening of a bend or buckle, the surface of the metal shall be tested for evidence of fracture.

SS 6-03.3(25) Welding and Repair Welding

Welding of structural steel shall be in accordance with the requirements in *Standard Specifications* Section 6-03.3(25). Welding will not be accepted as a substitute for bolting and should be done only where indicated in the plans. Adding even small welds not shown in the plans can induce high stresses in the members. This could seriously impair the strength and structural capability of the structure involved. The structure has been designed assuming that no additional welding will be done. The approval of the Assistant State Construction Engineer is required before doing any welding not shown in the plans.

Good workmanship and proper materials are essential. Welding operators should be qualified for the type of welding they are required to do. Welding procedures shall be reviewed by the Bridge Engineer before starting to weld on the structure.

Welding defects should be corrected as indicated in the *Standard Specifications*.

Low hydrogen type electrodes must be dry when used. The care and use of these electrodes as given in the *Standard Specifications* should be completely observed. No relaxation of these requirements can be tolerated.

SS 6-03.3(30) Painting

Steel structures shall be painted in accordance with the requirements in *Standard Specifications* Section 6-07.

SS 6-03.3(32) Assembling and Bolting

Before erection of the steel is commenced, the structural steel members shall be inspected for damage during shipping and handling. Any members that have been damaged must be repaired or replaced before being erected.

All members should have been match-marked and shall be assembled in accordance with the erection drawings from the Contractor. As the erection progresses, the Inspector should compare assembled members against the erection plans to see that proper members are in correct positions.

If during assembling, it is discovered that various members do not fit together, do not allow undue force to be applied to make them fit. The application of such a force can introduce stresses in several components of the structure. These stresses can be of a magnitude high enough to cause serious structural problems. The structure has not been designed to take these stresses. In such cases, the Assistant State Construction Engineer shall be informed.

Structural steel members that are improperly fabricated, or do not fit, shall be rejected and either repaired or replaced with new. If the Contractor elects to repair the structural member, the proposed repair procedure shall be reviewed by the Assistant State Construction Engineer prior to any repair work.

Unless otherwise shown or specified, structural steel connections shall be bolted. Simple truss spans shall be completely erected with all field-bolted connections and/or splices held in place with the minimum number of drift pins and bolts as specified in *Standard Specifications* Section 6-03.3(32). Once the minimum number of drift pins and bolts are installed in all the connections, final adjustments for span length and camber shall be made prior to completion of bolting and release of falsework. The assembly and bolting sequence for all structural steel structures shall strictly follow the erection plan. Erection and bolting sequences, especially cantilever and arch spans, are usually detailed in the contract documents.

Field connections shall be pinned and bolted in accordance with the requirements of *Standard Specifications* Section 6-03.3(32). This Section applies to connections and splices made in the field. Connections are when one structural steel member is bolted directly to another structural steel member; such as, cross-members and braces. Splices utilize structural steel plates to connect two structural steel members; such as, a plate girder. It also requires all connections and splices be securely drift-pinned and bolted before the weight of the member can be released or the next member is added. The field erection drawings must specify pinning and bolting requirements. *Standard Specifications* Section 6-03.3(32) then specifies the required minimum number of pins and bolts for field connections and splices.

Steel railings may be erected in place at the same time the trusses are erected but they shall not be finally aligned or bolted until after the concrete deck is placed. Railings shall be true to line, and for single spans shall show the camber of the span. For two or more spans the railing shall show a uniform camber over all of the spans; that is, the individual camber of each span shall not be carried in the railing.

SS 6-03.3(33) Bolted Connections

All bolted connections are designed by WSDOT to be friction connections. A friction connection transfers the stress by friction between surfaces in contact and does not depend on shear or bearing between members and bolts. The friction is provided when the connection or splice members are compressed through tension on the bolts (measured by turn-of-nut or direct-tension-indicator method). To develop design contact surface friction, all bolts in a bolted connection must be properly tightened to the minimum specified tension. The *Standard Specifications* recognize that final design loads are not present during erection of the structural steel members. Therefore, during erection, all the bolts are not needed in order to develop the friction necessary in the connection or splice for erection loads. The *Standard Specifications* recognize this and require a minimum percentage of the holes to be filled during erection; for instance, 50 percent for normal structures and 75 percent for cantilevered structures. These holes are filled with a combination of drift pins and bolts. Drift pins are required to properly align the members since bolts are usually smaller in diameter than the holes. Bolts are required to develop the minimum friction required to transfer erection loading. The minimum friction or load-carrying capacity is not developed until the bolts are tightened to the specified minimum tension.

Once the member is released from its support (support falsework or crane), the *Standard Specifications* specify the procedure required to complete bolting of each connection.

Sometimes fabricators will temporarily bolt-splice plates to the appropriate member. The fabricator will usually use the minimal number of bolts to secure the splice plate during shipping and handling. These temporary bolts shall be removed and replaced with high-strength bolts.

Structural steel field connections are made with high tensile strength bolts conforming to the requirements of *Standard Specifications* Section 9-06.5(3) and the special provisions. A special heat treatment gives these bolts a high tensile strength.

WSDOT designed bolted connections generally operate by a transfer of stresses by friction between surfaces in contact and do not depend on shear or bearing between the members and the bolts. Therefore, it is imperative that the contact surfaces of the metal shall be properly cleaned and the required minimum tension be obtained in the bolts.

The required tension in the bolts may be obtained by using either the Turn-of-Nut method or the Direct Tension Indicator (DTI) Method unless the specifications for the project state otherwise. If required because of bolt-entering and wrench operation, tightening by either procedure may be done by turning the bolt while the nut is prevented from rotating. *Standard Specifications* Section 6-03.3(33) requires a hardened washer under the turned element. Therefore, if the bolt is turned, a hardened washer is required under the bolt head. A hardened washer is also required with the DTI Method.

Bolted parts shall fit solidly together when assembled. Where an 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 to compensate for the lack of parallelism. See Figure 6-6. Bolts shall be tightened beginning from the center of each connection towards the edges of the connection. All joint surfaces, including those adjacent to the bolt heads, nuts or washers, shall be free of scale, except tight mill scale, and shall also be free of burrs, dirt, and other foreign material that would prevent solid seating of the parts.

Figure 6-6

	AASHTO M 164	AASHTO M 253
Type 1	A 325	A 490
	8S	10S
Type 2	A 325	A 490
	8S	10S
Type 3*	A 325	A 490
	8S3	10S3

*At the manufacturer’s option, Type 3 bolts may have additional distinguishing marks to indicate the bolt is atmospheric corrosion resistant and of weathering type.

AASHTO specifications require that bolts bear specific identification marks. The following identification is marked on the top of the bolt heads:

Nuts of all classes, in nominal diameter M5 and larger, shall be marked with the property class designation (5, 9, 10, 12, 8S, 10S, 8S3, 10S3) on the top or bearing surface, on the top of flange, or on one of the wrenching flats. Additionally, nuts of Classes 10, 12, 8S, 8S3, 10S, and 10S3 shall be marked with a symbol to identify the manufacturer. For Classes 8S3 and 10S3 nuts, the manufacturer may add other distinguishing marks to indicate the nut is atmospheric corrosion resistant and of a weathering grade of steel.

Type 3 bolts must be used when the structure is not being painted (WSDOT rarely utilizes unpainted structural steel for new structures). Nuts and washers used with Type 3 bolts must also have weathering characteristics.

Each fastener shall be tightened to provide, when all fasteners in the joint are tight, at least the minimum tension shown in the *Standard Specifications* for the size and grade of fastener used.

Turn-of-Nut Method

When the turn-of-nut method is used to provide the specified bolt tension, all of the required minimum number of bolts within a bolted connection or splice shall be brought to a “snug tight” condition. The bolts shall be tightened to “snug tight” in a systematical order to ensure that all parts of the joint are brought into full contact with each other. This usually requires that the bolts located near the center of the connection or splice be tightened first. Then all remaining bolts shall be tightened from the center progressing toward the outer edges. “Snug tight” is defined as the tightness attained by (1) a few blows from an impact wrench, or (2) the full effort of a man using an ordinary spud wrench. The “snug tight” requirement also establishes the starting point for full tensioning by the turn-of-nut method.

Once the bolts are snug tight, the outer face of the nut and protruding part of each bolt shall be match-marked with crayon or paint. The match-marking provides the control to both ensure the bolt does not rotate during tightening and measure the nut rotation. The required minimum nut rotation is listed in Table 4 of *Standard Specifications* Section 6-03.3(33). During this tightening operation, there shall be no rotation of the part not turned by the wrench.

Contractors often suggest a tightening method that eliminates marking the bolt as required in the turn-of-nut method. This suggested method requires calibration of the air impact wrench(es) and the inspection torque wrench. After calibration, the Contractor wants to snug tighten each bolt, then tighten to minimum tension using the air impact wrench without marking the nut and bolt. This method is heavily dependent upon the torque wrench test and is not accepted by WSDOT.

Direct Tension Indicator Method (DTI)

When the direct tension indicator method is used to provide the specified bolt tension, all of the required minimum number of bolts within a bolted connection or splice shall be brought to a “snug tight” condition. The bolts shall be tightened to “snug tight” in a systematic order to ensure that all parts of the joint are brought into full contact with each other. This usually requires that the bolts located near the center of the connection or splice be tightened first. Then all remaining bolts shall be tightened from the center progressing toward the outer edges. “Snug tight” is defined as the tightness attained by (1) a few blows from an impact wrench, or (2) the full effort of a man using an ordinary spud wrench.

This method uses a direct-tension-indicator washer that has formed protrusions on one face, leaving a gap. As the bolt is tensioned, the formed gap is reduced. The measurement of this gap verifies the bolt tension. *Standard Specifications* Section 6-03.3(33) addresses the maximum gap opening for direct tension indicators.

WSDOT has two concerns associated with the use of direct-tension-indicator washers. These concerns are (1) potential corrosion within the washer gap and (2) undetected bolt loosening as bolt tightening of a connection or splice proceeds. Following is a brief discussion of each item:

1. **Potential Corrosion** – The Specifications address this potential corrosion problem by limiting the maximum gap opening for painted and unpainted structures. These gap opening limits are governed by both tension requirement and required corrosion protection. The direct tension indicator manufacturers address only the minimum bolt tension requirement. It is, therefore, very important that the Inspector be aware of this additional concern of potential corrosion.

2. **Undetected Bolt Loosening** – The manufacturers of the direct-tension-indicator washers emphasize the ease and reliability of their product. They claim, and it is true, that if the gap is reduced to the specified maximum opening, the respective bolt is properly tensioned. The concern we have is that through the process of tightening all the bolts in a connection or splice, a warped plate may be progressively flattened, potentially loosening the initially tightened bolts. If this happens, the indicator washer still indicates the bolt(s) are fully tensioned. For this reason, WSDOT requires that bolt tension inspection, usually with a calibrated torque wrench, be performed. The Inspector should be aware of this potential problem and observe the tightening procedure with this in mind.

SS 6-03.3(33)B Bolting Inspection

The Inspector shall determine that the requirements of the *Standard Specifications* are met in the work. The Inspector shall observe the installation and tightening of bolts to determine that the selected tightening procedure is properly used and shall determine that all bolts are tightened and, in the case of the direct-tension-indicator method, that the correct indication of tension (gap) has been achieved. Bolts may reach tensions substantially higher than the value in Table 3 of *Standard Specifications* Section 6-03.3(33), but this is not cause for rejection.

The condition of the bolts is critical to the bolt-up operation and inspection. Bolts to be installed in the structure shall be lubricated in accordance with the *Standard Specifications*. A good check is a nut that is easily turned on the entire threaded portion of the bolt.

The following inspection procedure shall be observed for:

1. **Bolts tightened Using the Turn-of-Nut Method** – The Contractor, in the presence of the Engineer, shall use an inspection wrench which may be a torque wrench. Calibration of the inspection torque wrench is explained in a following section.

   Bolts that have been tightened using the turn-of-nut method shall be inspected by applying, in the tightening direction, the inspecting wrench and its job-inspecting torque to 10 percent of the bolts, but not less than two bolts, selected at random in each connection. If no nut or bolt head is turned by this application of the job inspection torque, the connection shall be accepted as properly tightened. If any nut or bolt head is turned by the application of the job inspecting torque, this torque shall be applied to all bolts in the connection, and all bolts whose nut or head is turned by the job inspecting torque shall be tightened and re-inspected. As an alternate, the Contractor may retighten all of the bolts in the connection, and then resubmit the connection for the specified inspection.

2. **Bolts Tightened Using the Direct-Tension-Indicator Method** –The Contractor, in the presence of the Engineer, shall use a feeler gauge to verify that each bolt has been properly tensioned to the maximum specified gap.

   If a bolt that has had its direct-tension-indicator washer brought to full load loosens during the course of bolting the connection, the bolt shall have a new direct-tension indicator washer installed and be re-tensioned. Reuse of the bolt and nut are subject to the provisions in the *Standard Specifications*.

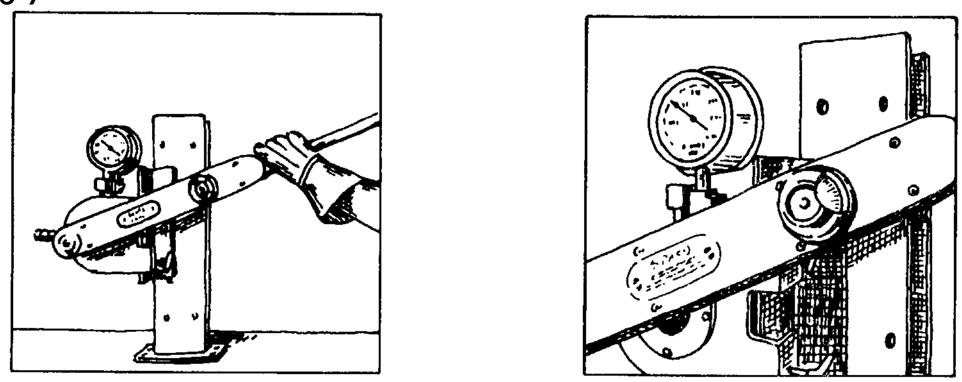
Calibration of Inspection Torque Wrench

Five bolts of the same grade, size, and condition as those under inspection shall be placed individually in a calibration device capable of indicating bolt tension at least once each working day. There shall be a washer under the part turned in tightening each bolt. Each bolt shall be tightened in the calibration device by any convenient means to the specified minimum tension. The inspecting wrench then shall be applied to the tightened bolt and the torque necessary to turn the nut or head 5 degrees (approximately 1 inch) at a 12 inch radius) in the tightening direction shall be determined. The job-inspection torque shall be taken as the average of three values, thus determined after rejecting the high and low values.

If the bolts to be installed are not long enough to fit in the tension calibrator, five bolts of the same grade, size, and condition as those under inspection shall be tested using Direct-Tension-Indicator (DTI) to measure bolt tension. This tension measurement test shall be done at least once each inspection day. 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 and the torque necessary to turn the nut 5 degrees (approximately 1 inch) at a 12 inch radius) in the tightening direction shall be determined. The job-inspection torque shall be taken as the average of three values, thus determined after rejecting the high and low values.

Figure 6-7 shows the operator calibrating a hand-indicator torque wrench. The bolt is brought to the proper tension by either method described above. The dial on the wrench was set at “zero” and sufficient torque applied to rotate the nut 5 degrees in the tightening direction. At this point, the wrench dial shows the kips required to further rotate the nut or bolt head. The torque wrenches used by inspectors of both the Contractor and WSDOT should be tested and compared at the same time for purposes of uniformity.

Figure 6-7



SS 6-03.3(35) Setting Anchor Bolts

Anchor bolts are usually plain round bolts with the head and plate washer on the lower end and the thread and nut at the top end. These bolts are set in pipe sleeves to allow room for adjustment of the span. Location of anchor bolt sleeves is very critical and must be verified by the inspector. Also, the exposed length of anchor bolts should be checked to ensure enough thread is exposed out of the pier cap to tie down the lower bearing assembly.

Anchor bolt sleeves, when anchor bolts will not be grouted until after freezing weather, must be protected against damage from expanded ice by filling the sleeves with ad nonevaporating antifreeze solution. Without exception, when piers and superstructures are constructed under separate contracts, the anchor bolt sleeves shall be filled with a nonevaporating antifreeze solution by the substructure Contractor. Before the bolts are grouted, the antifreeze solution shall be removed, the space well cleaned and the holes then filled with grout. The antifreeze solution shall be diluted with water and completely removed from the sleeves or it will have a detrimental effect on the filler grout. See *Standard Specifications* Section 6-02.3(18).

SS 6-03.3(36) Setting and Grouting Masonry Plates

It is important to set bearings level on all piers. Bridge plan bearing details usually show a leveling method. Bearings shall be set so that they are at zero movement at 64°F after the total load is applied and the span is released. The amount of offset varies with the length of the span and the temperature at time of erection.

Anchor bolt holes and the void underneath masonry plates shall be grouted, after all structural steel is erected and adjusted for length and camber, and at least seven days before the deck concrete is placed. Portland cement shall be used for grouting and the procedure should be as outlined in *Standard Specifications* Section 6-03.3(36).

Do not grout underneath masonry plates with dry mortar unless specifically shown in the plans. The Contractor shall build forms around the masonry plate about 4 in high and pour grout in the form from one side until the whole area is well filled. Use a wire or steel band to keep the grout flowing. After the grout has taken its initial set, remove the form and cut the edges of the grout with a trowel to about a 45 degree bevel from the bottom of the shoe to top of the pier. Do not allow the finished grout to extend above the bottom of the masonry plate.

SS 6-03.3(39) Swinging the Span

As required in *Standard Specifications* Section 6-03.3(39), the masonry plates shall be grouted and steel work, except railing, completely bolted and released from the falsework before forming for the roadway slab begins. Expansion dams shall not be bolted down until after the span is released from the falsework.

The camber diagram shown in the plans, especially for welded steel plate girders, quantifies the calculated deflection of the steel girder weight and the deflection of the girders due to the concrete slab weight. The camber diagram for the weight of the steel girders only is utilized by the girder fabricator.

Once all the temporary girder supports are removed, it is important that elevation control points on the top of the flanges of the girders or floor beams be established and permanently marked before any external load, such as form lumber, reinforcing steel, etc., is applied. These control points should be located at proper intervals to establish elevations for formwork and finished roadway slab grades. These control points should be at the span tenth points or at cross-frame locations (panel points).

Once these control point elevations are established, fills at each of these control points shall be calculated utilizing the camber diagram for the weight of the roadway slab and the profile grade. These control point fill values shall be used from that point on because it is extremely difficult, if not impossible, to calculate the deflection of the girders as formwork and reinforcing steel are added. These control point fill values will be used for the final adjustment of the roadway slab finish machine.

A pouring sequence for the roadway slab may be shown in the plans to reduce the size of the concrete pours, control deflection, and minimize tension cracking of the concrete slab during construction. Placing and finishing the concrete in the roadway slab shall be the same as for Concrete Structures covered in Section 6-2.

6-04 Timber Structures

SS 6-04.3(3) Construction Requirements

SS 6-04.3(1) Storing and Handling Material

Timber and lumber shall be stored off the ground and piled to shed water and prevent warping. Treated timber shall be handled carefully to prevent breaking of the outer fibers and rope or chain slings shall be used. Pike poles and peaveys are not to be used in handling treated timber.

All cutting, framing and boring of treated timbers shall be done before treatment insofar as is practicable. Framing shall be done in accordance with the requirements of *Standard Specifications* Section 6-04.3.

SS 6-04.3(3) Shop Details

Framing plans and details for treated timber structures shall be furnished by the Contractor and reviewed by the Project Engineer. After review of the framing details, one set shall be returned to the Contractor and one set furnished the shop inspector. Inspection of shop framing and treating of timber is performed by shop inspectors of the Materials Lab. Inspection reports showing details of treatment and lists of materials shipped will be mailed to the Project Engineer. Representative pieces of each shipment will be stamped by the shop inspector.

Untreated timber may be accepted on the basis of an inspection certificate in accordance with *Standard Specifications* Section 9-09.2(3).

SS 6-04.3(4) Field Treatment of Cut Surfaces, Bolt Holes, and Contact Surfaces

When field framing cannot be avoided, the cuts and holes shall be treated as required in the *Standard Specifications*. Timber for field treatment must be dry before applying the required treatment. Holes shall be bored for all bolts, drift bolts, boat spikes, dowels and truss rods using augers of the size specified in *Standard Specifications* Section 6-04.3(5).

After removal of temporary scaffolding and formwork, the nail and bolt holes in treated timber shall be repaired in accordance with the *Standard Specifications*.

Field treatment for structures of untreated timber shall be in accordance with the requirements in *Standard Specifications* Section 6-04.3(4).

SS 6-04.3(18) Painting

Painting of timber structures shall be in accordance with the requirements in *Standard Specifications* Section 6-07.

6-05 Piling

GEN 6-05.1 Vibration Monitoring during Pile Driving

On some projects, pile driving vibrations will be monitored for potential damage to adjacent structures or buildings. When that monitoring indicates a potential for damage, the Project Engineer should ensure that the minimum size hammer specified for the piling being driven is actually being used. If so, and vibrations are still potentially damaging, the State Construction Office should be notified to determine if preboring or jetting should be used to reduce vibrations. Should preboring or jetting, or other methods be determined necessary, such work shall be considered a change in accordance with *Standard Specifications* Section 1-04.4.

GEN 6-05.2 Pile Driving Records

Pile driving records are to be kept in the Pile Driving Record Book (DOT Form 450-004) or on the Pile Driving Log (Form 450-004A), which becomes part of the project final records. This book has sufficient room for a condensed pile driving history, pile layout, and miscellaneous notes in addition to the driving log for each pile. Number the piles on the sketch in the pile layout and use these for the Pile No. on the pile driving log.

The pile driving record book contains instructions for completing the driving log. In order for this log to furnish complete information on the pile driving work, it is imperative that it be filled out completely in accordance with the instructions in the book. If more space is necessary, use more than one page for the pile. Items in the heading which are the same for several piling, may be marked “Same as Pile No. ___.”

The piling should be marked every foot of their lengths with crayon or paint unless there is some other method of determining when each foot of the pile has been driven. Count and record the number of blows per foot and hammer energy as the pile approaches bearing.

SS 6-05.3 Construction Requirements

SS 6-05.3(2) Ordering Piling

Piling shall conform to the requirements of *Standard Specifications* Section 9-10. When piling is received on the project, it shall be inspected and a notation made in the Section of Miscellaneous Notes in the Pile Record book. Untreated timber piles will be inspected in the field and accepted for use there. All other piling, except concrete piles cast on the job, will be inspected by Fabrication Inspectors before delivery.

The lengths of piling required are determined by driving test piles or by other information which may be available. The Project Engineer provides the Contractor with an order list for timber and precast concrete piles. This list must show the length of piles required below cutoff (the top of the pile within the footing). The Contractor should be informed that the lengths shown on the order list should be increased, at their expense, the necessary amount to provide for fresh heading and to reach from the cutoff elevation up to the position of the driving equipment. Payment for piling will be made for the number of feet shown on the order lists except that if greater lengths are driven, with the concurrence of the Project Engineer, payment will be made for the lengths actually driven below cutoff. Itemized lists for cast-in-place piles or steel piles will not be furnished by the Engineer.

SS 6-05.3(3) Manufacture of Precast Concrete Piling

SS 6-05.3(3)A Casting and Stressing

Curing beds for steam cured concrete piles shall not rest directly on the floor but shall be elevated enough to permit the complete circulation of steam around the piles.

Lifting loops shall be removed to ½ inch below the surface of the concrete and the hole filled with mortar.

Concrete piles shall be handled as described in the *Standard Specifications*, the *Standard Plans*, or as shown in the plans in order to avoid excessive deflections and strains.

SS 6-05.3(6) Splicing Steel Casings and Steel Piles

When steel piles must be spliced and splicing details are not shown in the plans, the splice should be made with a single V-butt weld over the whole cross-sectional area of the pile. Welding shall be done with specified welding rod and suitable equipment in accordance with American Welding Society Specifications and good industry practice. A qualified welder is required. See *Standard Specifications* Section 6-05.3(6).

No Engineer's order list will be given for steel piling.

SS 6-05.3(7) Storage and Handling

SS 6-05.3(7)A Timber Piles

Chain slings will be permitted in handling treated timber piles. Treated timber piling shall be furnished and driven full length, i.e., without splices. The entire length shall be pressure treated. Therefore, the pile tip shall not be cut after treatment. If splices become necessary and the order length furnished by the Engineer is insufficient, the State Construction Office should be contacted for direction. However a splice probably will not be considered if it cannot be located below the permanent water table elevation.

SS 6-05.3(7)B Precast Concrete Piles

Precast concrete piles require special care in storage and handling, especially when raising them into the leads. The general method of attaching slings for handling is described in the *Standard Specifications*. Long piles must be supported at the ends and at intermediate points to prevent undue bending and cracking of the concrete. In special cases the plans may show the method for lifting long piles. Some pile driving crews lack experience with concrete piles and handle them as they are accustomed to doing with timber piles. Such handling will probably result in damage to the concrete piles and must not be allowed.

SS 6-5.3(7)C Steel Casings and Steel Piles

Steel piling shall be handled in such manner as to prevent bending of the flanges, and when stacked they shall be supported in such a manner that the piles will not bend.

No Engineer's order list will be given for cast-in-place concrete piling.

SS 6-05.3(9) Pile Driving Equipment

SS 6-05.3(9)A Pile Driving Equipment Approval

The type and size of hammers to be used to drive piling are specified in *Standard Specifications* Section 6-05.3(9)B. The Project Engineer shall require the Contractor to furnish full information on any hammer proposed for use so it can be determined whether or not the hammer meets the requirements of the specifications and that the bearing capacity of driven piles may be computed. It is very important to verify that the drop of the ram is in accordance with the submitted data. Otherwise, the pile bearing calculations will not be correct. A useful formula to determine the drop of a single acting diesel hammer determined from measuring the blows per minute is:

Stroke Formula (ft of drop) = (4.01((60/BPM)2)-0.3)

Where BPM is the blows per minute of the hammer.

This drop can then be used in the bearing equation shown in *Standard Specifications* Section 6-05.3(12) to determine the bearing of the piling.

This formula calculates the drop from the rate of blows per minute that the hammer is hitting at and makes it no longer necessary to watch the top of the hammer and estimate the distance that hammer is coming out of the casing. Since the rate the hammer runs at is dependent on the drop of the hammer, and this hammer drop is accelerated at a constant by gravity, the distance the ram travels can be determined from the formula.

SS 6-05.3(9)B Pile Driving Equipment Minimum Requirements

Standard Specifications Section 6-05.3(9)B and the special provisions, govern the hammer size by specifying the minimum ram weight and the minimum energy required for each type of pile, required bearing, and hammer. The most commonly used hammers are air, hydraulic, or diesel activated. The hammer energy output is simply the weight of the ram times the distance the ram falls. This energy determination is a simple matter with a drop, hydraulic, or air/steam activated hammer. The measurement of the energy output of a diesel activated hammer is more complex. The minimum energy required by the specifications is the energy output of the hammer at the point of impact at the required pile bearing. The hammer needs to operate at or above the required minimum energy level in order to achieve the specified pile bearing capacity.

The Project Engineer may concur with the Contractor's proposed hammer if it meets the criteria of the *Standard Specifications* and the special provisions. During field operations, the pile driving hammer must be capable of delivering at least the required minimum energy at the required pile bearing value. The State Construction Office should be consulted for any other hammer submittals or insufficient performance in the field.

Drop hammers, which are rarely used, must be weighed, in accordance with *Standard Specifications* Section 6-05.3(9)B, before any piles are driven. The drop hammer stroke should be carefully measured. This can be done by taping a piece of rope or rag around the hammer line at the height above the hammer for the drop desired. The hammer operator can then gauge the drop with reasonable accuracy. The stroke (drop) of the hammer ram must be consistent with the required minimum energy.

Air or steam activated hammers lift the ram by either air or steam pressure to a predetermined distance and release the ram. The energy is produced by the falling ram. These hammers usually operate at 50 to 60 blows per minute depending on the hammer manufacturer. A count of the actual blows per minute will provide verification that the hammer is operating properly. If the blows per minute exceed the published manufacturer's data sheet for the specified minimum energy, and the Contractor is not able to find and rectify the problem, the State Construction Office shall be notified. No additional piling are to be driven until the problem is resolved.

Hydraulic activated hammers lift the ram by hydraulic fluid pressure to a predetermined distance and then release the ram. The energy is produced by the falling ram. There are two types of hydraulic activated hammers, single and double acting. The hydraulic activating systems for both of these types of hammers are totally enclosed using a vegetable oil medium, rendering them environmentally friendly. The method for measuring the energy output is different for each type of hydraulic activated hammer. The energy output for each type can be varied by using simple adjustment procedures. Again, the respective hammer must be operating at or above the specific minimum energy when the required pile bearing capacity is reached.

Diesel activated hammers lift the ram by energy produced when diesel fuel is ignited. The energy produced is a combination of the fuel explosion and the drop of the ram. There are two types of diesel activated hammers, single and double acting. The method for measuring the energy output is different for each type of diesel activated hammer. Diesel hammers produce a variable energy. The variable energy output of a diesel hammer is dependent on a number of factors, which include fuel quality, fuel setting, soil conditions, and resistance from the pile being driven. As the pile resistance increases, the energy output of a diesel hammer usually increases. The manufacturer's maximum energy value for each diesel hammer is measured in the laboratory using a hammer in tip top shape. For this reason, it is a good idea to have a hammer on the project with a maximum rated energy higher than the contract minimum required energy. A good rule of thumb when selecting a diesel hammer is that, if 80 percent of the maximum energy of a hammer equals the contract minimum required energy, the diesel hammer will produce sufficient energy to meet the contract energy requirements.

A single acting diesel activated hammer is open at the top, and at the top of the ram stroke a portion of the ram is usually visible. The bearing value of the pile being driven is determined by the number of blows per foot at a blows per minute rate. The energy output of a single acting diesel hammer is determined by the blows per minute of the running hammer. The manufacturer is required to submit this energy data. The rate (blows per minute) is dependent on how high the ram raises up (stroke) due to the diesel fuel combustion. Thus, the longer the stroke, the greater the energy and the longer it takes. In other words, as the rate (blows per minute) decreases, the energy output increases.

A double acting diesel activated hammer is closed at the top. This closed top acts as a pressure chamber driving the ram back down where the diesel fuel explosion occurs. The bearing value of the pile being driven is determined by the number of blows per foot at a measured pressure within the top bounce chamber. The energy output of a double acting diesel hammer is determined by the measured bounce chamber pressure while the hammer is operating. The manufacturer is required to submit this energy data. Each double acting diesel hammer comes with a hose running from the bounce chamber to a box containing a pressure gauge. There is usually a button on this pressure gauge box. When the button is depressed the gauge is activated with the bounce chamber pressure. If this button is depressed continuously, the hammer efficiency decreases because of the pressure bleed off created by the pressure gauge operation. The button should only be depressed periodically when an energy reading is required. The pressure reading and corresponding energy shall meet the minimum energy at the required pile bearing value.

The Contract allows the use of vibratory hammers to initially set piles. As of yet, there is no reliable means of determining the actual bearing capacity of a pile driven by a vibratory hammer. Often, the contractor wants to initially set piles with vibratory hammers if the soils and/or limited access are such that impact hammer operation would be difficult. The Contract allows this but requires that an impact hammer be used to acquire the bearing capacity. Since static friction is usually much higher than dynamic friction, the actual bearing capacity is determined while the pile is in motion. This requirement is governed by the contract requirement that the pile must be driven at least an additional 2 feet using an impact hammer with the blow count (blows per inch) constant or increasing. If the contractor uses a vibratory hammer to initially set the piles, there must be a comprehensive procedure to ensure proper location an plumbness of each pile. This is usually accomplished by providing a rigid steel template and using good conscientious control while setting and initially driving each pile.

SS 6-05.3(9)C Pile Driving Leads

Pile driving leads shall be fixed at the top and bottom as discussed in *Standard Specifications* Section 6-05.3(9)C, to ensure that the piling can be accurately driven both as to position and batter.

SS 6-05.3(10) Test Piles

A careful study should be made of the foundation exploration data shown in the plans and/or included in the Geotechnical Report before driving any test piles. Care should be taken that the test piles are not stopped on a relatively thin hard layer overlaying softer material. After the test piles have been driven, an effort should be made to correlate the results with the foundation data before ordering the permanent piles. The results from driving the test piles should be discussed with the Regional Operations/Construction Engineer if they do not correlate with the foundation data.

Test piles shall be driven to at least 15 percent more than the ultimate bearing capacity 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 ultimate bearing capacity as the production piles. Test piles shall penetrate at least to any 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 groundline, and 16 feet below the bottom of the concrete seal.

Preboring, jetting, or other means may be used to secure minimum penetration with the test pile if such means is necessary and will be used for the permanent piles. The reason for driving the test pile is to obtain information for ordering the permanent piles, and to obtain additional information relative to driving the permanent piles.

It is the responsibility of the Contractor to supply test piles of sufficient lengths to provide for variation in soil conditions. If the piles furnished are not long enough, or are unsuitable in other ways, it will be necessary for the Contractor to supply acceptable piles. Followers will not be permitted in driving test piles. A follower is a member interposed between a pile hammer and a pile to transmit blows while pile head is below the reach of the hammer (pile head below the bottom of leads).

The State Construction Office should be notified of the date test piles will be driven.

Test piles shall also be recorded in the pile driving record book. In addition, following the driving of each test pile, the Test Pile Record form shall be completed and sent to the appropriate offices the following day. This form should be filled in completely, including the rate/pressure of the hammer. Record the bearing value of the test pile for each foot as it is driven.

SS 6-05.3(11) Driving Piles

It is suggested that the State Construction Office be contacted before any piling are driven.

SS 6-05.3(11)A Tolerances

Foundation piles must be driven true to line and in their proper position so that full bearing and lateral support is secured for each pile. Each pile has been definitively positioned in the design, and piles should be driven as nearly as practicable to the position shown. Any variation of 6 inches or more from the plan shall be reported to the State Construction Office before accepting the pile. The tolerance for all types of battered piles is ¼ inch in 12 inches. Any deviation exceeding this tolerance shall be reported to the State Construction Office for evaluation.

Care shall be taken in driving steel H piles to ensure that the driven pile is oriented as close as possible to that shown in the plans. Pile design usually involves horizontal forces due to temperature, concrete shrinkage, earthquake, and wind as well as axial forces, and if a driven pile is not aligned as shown in the plans, the pile may become overstressed due to excessive bending stresses. Any deviation of more than 20 degrees from the pile axis or more than 6 inches from the position shown in the plans shall be reported to the State Construction Office for evaluation and acceptance.

Large diameter prestressed concrete cylinder piles are not completely covered in the *Standard Specifications*. The requirements of the special provisions must be observed. Accuracy of placing and driving is most important. Every effort should be made to prevent these piles from drifting out of line or out of plumb during driving, but care must be taken to avoid applying excessive lateral force which may crack the pile. These piles do not have to be very far out of plumb before excessive overstress occurs. When a driven pile is found to be cracked or is out of plumb, it should be referred to the State Construction Office for a decision regarding corrective action to be taken.

SS 6-05.3(11)D Achieving Minimum Tip Elevation and Bearing

Piling shall be driven to develop the bearing value as shown in the plans or in the *Standard Specifications*. The penetration of the piles under the last few blows must be carefully gauged and the bearing value computed by use of the formula shown in the *Standard Specifications*. Pile driving specifications should be administered with a great deal of common sense. There is no substitute for experience and good judgment.

Often the foundation reports contain two pile tip elevations, “estimated tip” and “minimum tip” elevations. The estimated tip elevation is simply the elevation that the tip is estimated to be driven to and is utilized to determine driving length quantities in the bid item for furnishing piling. Minimum tip elevations are often specified in the contract plans. These are usually to ensure that piles do not hang up on logs, a thin hard soil layer and other obstructions, or to achieve a minimum pile penetration (e.g., uplift and/or lateral load capacity). Minimum tip elevations are also specified where resistance to uplift is taken into consideration in the design of the foundation seal thickness. The minimum tip elevations should be higher than the estimated tip elevations. The Project Engineer should always review the tip elevations in the plans and compare them to the foundation report recommendations. Any discrepancies should be reported to the State Construction Office.

The minimum tip elevations is a design parameter that may come from the geotechnical design or the structural design. A pile tip elevation that is less than minimum cannot be accepted in the field, it must be reviewed by the State Bridge and Structures Office, the State Bridge Construction Office, and the State Geotechnical Engineer. If, during the initial pile driving operations, minimum tip is not being achieved, no additional piling should be driven until concurrence is obtained to change the minimum tip elevation, or the contractor will have to change his method of installation so that the minimum tip elevation can be achieved.

The use of water jets may be required for driving piles, especially for concrete piles. The piles must be driven at least 6 inches after the jet is removed, or to the required bearing. Do not allow the nozzle of the jet to penetrate below the tip of piling previously driven. Mark the jet pipe in such manner that the operator and Inspector can determine the depth required. The State Construction Office should be notified if water jets are proposed for use.

Preboring may also be used to secure the minimum specified penetration. Usually the prebored hole should be slightly smaller in diameter than the pile and the depth of preboring should be less than the minimum specified penetration. However, conditions may exist which make it necessary that a larger hole be prebored and the space around the pile be filled with sand while the pile is being driven to the specified bearing. Unless water-jetting, preboring, or other means of securing minimum penetration is specified and payment is provided for in the contract provisions, this work will be at the Contractor’s choice and expense. However, the procedure used must be reviewed by the Engineer and shall result in a satisfactory pile and will not damage the integrity of the structure, roadway, adjacent structures, or utilities. Any damage done must be repaired to the satisfaction of the Engineer at the Contractor’s expense.

Where the specified minimum tip elevations cannot be reached the State Construction Office shall be notified.

SS 6-05.3(11)F Pile Damage

Rejected piles shall be removed or cut off 2 feet below the bottom of the footing. Rejected casings for cast in place piles that are left in place shall be filled with sand.

In driving precast concrete piles, several layers of plywood or a 3½ inches wood block should be placed between the top of the pile and the steel driving head of the hammer. Care should be taken to prevent crushing of the pile head before the desired penetration is reached. Where crushing occurs, the top of the pile should be checked to determine if the end is square with the body of the pile; also, the hammer should be checked to determine if a fairly flat blow is being delivered to the pile. In driving concrete piles, it may be advisable, in order to prevent crushing of the head and to obtain the required penetration, to operate a hammer at less than full throttle until just before completing the driving, after which the throttle should be fully opened in order to obtain the true bearing value of the pile.

SS 6-05.3(13) Treatment of Timber Pile Heads

The handling and driving of treated piling require special care. Heads of piles should always be freshly cut, and rings or wire mesh screens placed on top during driving. In wet weather the final cutoff should be at least 1 foot long and the creosote, pitch and fabric cover placed immediately after the pile is cut. Do not make a cutoff and then wait until the next day to place the cover. Fabric covers should be well tacked to the pile and neatly trimmed to within 3 inches of the top of the pile so that the fabric will not have ragged edges. A follower driving cap should be used on treated piles. This is to help hold the pile in line to minimize the use of chocks in the leads during driving. Timber piles must be strapped in accordance with the requirements of *Standard Specifications* Section 9-10.1 before they are driven.

SS 6-05.3(15) Completion of Cast-In-Place Concrete Piles

The casings for piles cast in place shall be carefully checked after driving, for water tightness and deformation of the casing due to the driving of adjacent piles. A mirror for reflecting light into the casing is the most common method for this check. On cloudy days, a flashlight may be lowered into the casing.

Immediately after driving, the pile casing shall be covered to prevent dirt and water falling into it. All debris and water shall be removed from the casing prior to placing the reinforcing steel cage. No water will be permitted in the casing when concrete is placed.

Due to the ever increasing loading from earthquake activity, most cast in place piling require reinforcement for the full depth of the pile. This full depth reinforcement presents extreme difficulty in placing concrete with a rigid conduit the full depth, especially if the pile is battered. For this reason, Class 5000P concrete is required. This class of concrete has small aggregate and fly ash making the mix rather sticky and cohesive, which reduces the likelihood of segregation during placement. This concrete shall be placed continuously through at 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 foot 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.

6-06 Bridge Railings

GEN 6-06.1 Railing Alignment

Railings shall be carefully aligned, both horizontally and vertically, to give a pleasing appearance. On multiple span bridges, the rail and wheel guard or curb heights at the ends of each span should be varied a sufficient amount to produce a uniform camber or grade from end to end of the bridge.

At the beginning and ends of horizontal curves and through vertical curves, the height of curbs may need to be varied so that the rail heights will be uniform above the curb. On any structure on which occurs a break in grade, horizontal curve with superelevation, vertical curve, or a combination of the three, the Project Engineer should plot to a large scale, the profiles of the roadway grades at the curb lines. From these profiles the grades for the tops of the curbs and railings can be properly determined. A slight hump in the rail over the whole structure is usually not objectionable, but a hump and then a sag is not permissible.

6-07 Painting

GEN 6-07.1 General

When inspecting bridge painting for steel structures, the Inspector should prepare a plan for the structure they will be inspecting. This plan will enable the Inspector to locate sections of the structure where painting activities occurred.

An Inspector's Daily Report should be filled out after every work day with the activities performed and related to the Inspector's bridge plan. In the daily report, the Inspector should identify the activities such as cleaning, blasting, and applying the base, intermediate, and finish coats. These daily reports should accurately represent the work accomplished and any noted deficiencies.

The Inspector should become familiar with the latest safety requirements. Contract environmental requirements should be reviewed as well.

Manufacture and shop mixing of paint materials are controlled from the State Materials Laboratory. Each container in each shipment of paint should bear a lot number, date of manufacture, type of paint and manufacturer's name.

When quantities of paint required for a particular job are 20 gallons or less, they may be manufactured and shipped without inspection and testing by the laboratory. A certificate of compliance with specifications signed by the manufacturer shall be presented to the Project Engineer by the Contractor at the time the paint is brought to the project site.

All paint shall be thoroughly mixed before using. Paint may be mixed by stirring with hand paddles or by using power stirrers.

All paints bearing dates of manufacture over one year old should be sampled on the basis of one sample per batch. Paint showing appreciable deviation from normal should be sampled and set aside until checked and released by the State Materials Laboratory.

The paint should be capable of application at the required thickness without any sags or runs. If it is not possible to do this, the State Materials Laboratory should be contacted for necessary steps to be taken.

SS 6-07.3(9) Painting New Steel Structures

SS 6-07.3(9)I Application of Field Coatings

New steel, shop coated before erection, shall have all erection and transportation scars, rivet heads, and welds cleaned and spot coated. If a dirt film has accumulated on the steel during the erection period this must be removed by flushing. All concrete residue must be removed from the floor system after the deck pour is completed. Generally, this may be accomplished by flushing before the residue has set up and while the pour is in progress.

All coatings shall be applied per the manufacturers recommendations.

Brushes and spray equipment should be in good condition. An intermediate stripe coat should be applied to the metal edges, inside angles, welds, bolt heads, nuts and rivets prior to the application of the full intermediate coat of paint. The use of inspection mirrors is required for reflecting light into the interior of boxed sections or members for locating painting defects.

The Inspector must check to see that the proper film thickness of paint is applied. Wet film thickness is to be measured immediately after the paint is applied and the dry film thickness is to be measured after the paint has become thoroughly dry and hard. It is difficult to measure the dry film thickness of paint on galvanized metal so it is necessary to measure the wet film thickness for each coat of paint as it is applied.

When an Inspector finds an area where the painting does not meet the specifications, they should mark the area with contrasting brightly colored alkyd paint from an aerosol can. A light coat of this spray paint will not adversely affect the paint job and it will effectively mark the area to tell whether correction work was performed on the area. Marking the area with spray paint provides the Inspector with an easy method of marking deficient areas and provides the Contractor a ready method of locating the areas that require additional work. This will also free the Inspector to concentrate on areas of serious deficiencies without losing control over those requiring minor corrections. When marking the final coat, be careful to mark only the area to be reworked.

Adequate staging, scaffolding, ladders, and fall protection are required to be provided by the Contractor to ensure safety to workmen, room for good workmanship, and adequate facilities for proper inspection.

Technical assistance and equipment are available at the State Materials Lab, and on request can be provided at the job site to ensure a good paint job.

SS 6-07.3(10) Painting Existing Steel Structures

SS 6-07.3(10)A Containment

Containment systems are required by the Contract. Containment systems are required during the cleaning and painting of the bridge. These systems are necessary to prevent contaminants from entering state waters.

SS 6-07.3(10)D Surface Preparation Prior to Overcoat Painting

Cleaning for removal of rust or corrosion spots in repainting and cleaning of new steel shall mean "commercial" abrasive blasting as defined in the *Standard Specifications* or the special provisions.

Wire brushing and scraping shall normally be limited to removal of dirt and loose paint where corrosion is not involved.

All rust which cannot be removed by abrasive blasting shall be removed with chisels, hammers or other effective means as directed by the Engineer.

When called for in the *Standard Specifications* or the special provisions, the entire structure shall be pressure flushed with water from the top down before other cleaning or painting is started. The nozzle should not be more than 9 in from the surface being cleaned. A biodegradable detergent may be added to the water jet to remove oil and grease. Biodegradable detergents shall be reviewed by the State Materials Laboratory and precautions taken to avoid harmful residue on the steel.

In addition to the initial pressure flushing, all abrasive blasting residue must be removed after blasting and spotting and before application of additional paint. Pressure flushing may be required for this purpose if the Project Engineer deems it necessary.

On repainting projects, the Engineer or Inspector should observe and report to the State Bridge and Structures Engineer any spot or area where corrosion or other deficiencies are of such extent as to threaten the strength of the steel member. They should also observe areas where water becomes trapped to ultimately endanger the steel through corrosive action, and advise the Regional Operations/Construction Engineer, so the condition may be corrected.

SS 6-07.3(10)F Collecting, Testing, and Disposal of Containment Waste

During the preparation and painting of steel bridges, it is very important that the Inspector be aware of the potential impact to the surrounding environment. The air, water, and land quality are of major concern. WSDOT and environmental agencies are working together to establish guidelines for bridge painting. Policies and procedures involving environmental concerns will be addressed in the contract. Compliance to these specifications should be closely monitored.

Many bridges that are being repainted have been previously painted with lead based paint. When this is the case, the Contractor must submit a "Lead Health Protection Program" (WAC 296-155-176). The waste generated from cleaning the bridge (bird guano, paint chips, etc.) must be tested as outlined in the contract provisions. Handling and disposal of this wasted must be as prescribed by current state law. Contact your Regional Environmental Office regarding disposal of lead paint waste.

The protection of the structure, traffic, and property from splatters and airborne paint spray is the responsibility of the Contractor. Since WSDOT may be criticized because of damage from paint, the Engineer must enforce the provisions of the contract to ensure protection therefrom.

6-08 Bituminous Surfacing on Structure Decks

GEN 6-08.1 Description

Most paved structures have a BST or HMA philosophy that manages the asphalt depth economically in the long term. The intent of the management is to protect the structures from excessive pavement weight, and minimize the risk of equipment loads and planer damage. Therefore, Section 6-08 addresses structural paving issues not addressed in Division 5.

SS 6-08.2 Materials

The intended use of Bridge Deck Repair Material in Section 9-20.5 is for deck patching prior to placing a membrane.

SS 6-08.3(2) Contractor Survey for Grade Controlled Structure Decks

The Plans specify Grade Controlled or Not Grade Controlled for each structure. This information is necessary for the Contractor QC and WSDOT QA, if desired. A Grade Controlled structure requires a Contractor survey of the existing grade profile and includes measurement of the asphalt depth prior to pavement removal when removal is to be achieved by rotary milling/planing. The Contractor needs to know the existing planing depths, in advance, to avoid damaging the concrete. The Contractor needs to know the Final Grade Profile for tolerance acceptance.

When scraping is the method of Full Removal, the asphalt depths do not need to be known prior to removal.

The Project Engineer must review the Contractor survey for safe planing depths and adjust the Final Grade Profile to meet the desired uniform depth specified in the Plans. Adjusting the Final Grade Profile and planing depths should consider the following:

1. Contractor survey for removal: Submittal review should always assume the existing asphalt depths were unknown or inaccurate at the Design stage and use the measured depths provided in the survey. Grade Controlled, Partial Removal milling depth should not be within 0.10 foot or 1¼ inches of the concrete structure at any location to preserve the deck and membrane.

   Full Removal planing should not contact the concrete deck surface. Prior to milling, the Project Engineer must check the asphalt depth to the original concrete surface at all locations. The maximum mill depth should be as close as possible or to within 0.01 foot or ⅛ inch of the top of the existing deck, and not below. The planed surface should be uniform, flat, and not remove the concrete. Ideally, the asphalt removal exposes the deck without removing a layer of concrete rebar cover. Practically, there will be areas of thin pavement in previous rutts, and areas of over milling or damage. In these areas, the asphalt depths will vary and appear as inconsistent data because the original grade has changed. Using the Contractor survey, it is up to the Project Engineer to determine original grade and safe removal depths. Profile changes are often undocumented and buried on structures that have been widened in the past. Excessive pavement depth also contributes to inconsistent surveyed data.

2. Contractor survey for Final Grade Profile: The survey includes the profile beyond the structure for two reasons:
   a. To provide a smooth transition from the existing roadway to the structure profile grade.
   b. To identify existing problems in the transition zone.

   If there is a grade profile problem on or off the structure, the Project Engineer should address the Final Grade Profile adjustment with the Contractor. If it is necessary to raise or lower the Final Grade Profile, or transition the grade, the maximum rate of grade adjustment or slope is 0.2 percent (1'/500'), per Standard Plan A60.30. Skewed bridge ends, cross slope transitions, and significant summit or sag vertical curves require extended grade transitions.

   If previous paving has not been transitioned smoothly (too short), the length of the transition must be extended. If extending the transition places it outside of the project limits, contact the HQ Construction Office. An improper transition is unacceptable for two reasons:
   a. This is the common cause of many “bumps at the bridge” and may reduce the load rating. The Bridge Office may have to restrict truck loading if the transition is bad enough.
   b. It is a waste of Maintenance resources to place a temporary wedge patch to address smoothness.

SS 6-08.3(4) Partial Depth Removal of Bituminous Pavement from Structure Decks

Grade Control applies to Partial Removals when a grade correction is required on or off the structure. Partial Depth or Mill/Fill planing should never contact the concrete deck surface, see SS 6-08.3(2)1, Paragraph 1. Milled areas which contact the concrete deck or membrane should be marked for repair as damaged concrete and require a membrane repair.

SS 6-08.3(5) Full Depth Removal of Bituminous Pavement from Structure Decks

Prior to milling operations, the Inspector must verify the rotor head ¼ inch tooth spacing and tooth length tolerance. Common planer tooth spacing of ⅝ inch or planer teeth that are not uniform length provide a surface that is too rough for waterproof membranes. Planer teeth that are worn down and not sharp severely damage concrete.

Remove loose, unbonded, or substandard HMA prior to placing a membrane. HMA in good condition and firmly bonded to the concrete does not have to be removed or chained. A Chain Drag applies to the remaining area of exposed concrete to identify repairs.

SS 6-08.3(6) Repair of Damage due to Bituminous Pavement Removal Operations

Full Removal planing must be uniform and smooth where occasional tooth strikes in the deck are unavoidable within the planing tolerance.

Milled areas below the maximum mill depth tolerance should be marked for repair as damaged concrete.

Planer damage consisting of concrete edges and ridges should be repaired flush to grade with a grout material to avoid stretching or tearing the membrane when HMA is compacted. Do not pay for this work in the Bridge Deck Repair item.

SS 6-08.3(7) Concrete Deck Repair

Standard Plan A-60.40, "HMA Overlay Further Deck Preparation" is available for reference and details of Bridge Deck Repair.

A qualified Region Materials staff or Inspector must be available during or shortly after the removal process in order to complete a Chain Drag test timely. The chaining identifies the existing Bridge Deck Repair quantity on the structure, whereas the Plan quantity for the Bridge Deck Repair item is an estimate that limits the cost risk and closure time to the contract. Administration of Bridge Deck Repair should follow these guidelines:

1. Section 1-04.6, Variation of Estimated Quantities applies for payment since the Chain Drag quantity and the Plan quantity will seldom match.
2. If the Chain Drag testing indicates more repairs than the Plans, it is preferred but not mandatory to negotiate more or all of the repairs within the contract. If the chained quantity exceeds 125 percent of the Plan quantity, contact the Bridge Office and the HQ Construction Office for a recommendation to proceed because excessive Bridge Deck Repair may not be cost effective and concrete deck rehabilitation may be required.
3. If the contract cannot complete all repairs, the priority repair areas are:
    a. Full depth repairs or holes in the deck
    b. Areas with exposed rebar to protect the steel
    c. Fill in spalled areas to provide a level surface for the membrane
    d. Delaminated areas
4. The Project Engineer must submit the Chain Drag Report spreadsheet to the Bridge Deck Program Manager in the Bridge and Structures Office in order to manage the concrete deck needs statewide. The Chain Drag Report spreadsheet can be obtained on the State Construction Office SharePoint site.

A Chain Drag Report documents the deck conditions after Contract. The spreadsheet has instructions to document the area (square foot) of patches, spalls, delaminations and other defects. The primary function of the report is to describe the total patching completed in the contract, and to note any deck defects or paving construction issues for future reference. The secondary function is to document the amount of incomplete repair, which is the basis for estimating the future Bridge Deck Repair quantities in the next Full Removal.

If rotary milling exceeded the depth tolerance and damaged the concrete, these areas are marked for repair at the contractor's expense in accordance with Section 6-08.3(6), Repair of Damage due to Bituminous Pavement Removal Operations.

SS 6-08.3(8) Waterproof Membrane for Structure Decks

The Contractor must install the Bridge Deck Waterproof Membranes in accordance with the manufacturer's recommended products and installation documents. Primers must cure or the membrane may not stay in place during compaction. At night, a hand held spotlight will show a dull finish when the primer has cured vs shiny when wet. Cooler temperatures or higher humidity will take longer to cure. Inspect the membranes during placement for construction defects that poke holes during compaction; and while paving to ensure the paver does not drag on the membrane or other equipment does not tear the membrane with turning movements.

SS 6-08.3(9)A Protection of Structure Attachments and Embedments

Bridge expansion joints vary in size, materials and complexity. Contractor placement operations must not leave BST or HMA in expansion joints. The Contractor shall remove all materials dumped through the joints to the substructure. Bridge Maintenance is not funded to clean up or repair this contract work.

SS 6-08.3(11) Paved Panel Joint Seals and HMA Sawcut and Seal

The Contractor must mark the locations of the exact ends of sawcut for a string line before paving unless there is a gap in the bridge curb clearly indicating the location. Usually, it will be difficult to find after paving and sometimes the gap in the curb does not line up with the expansion gap. Watch for joints that have a jog or are not a straight line from curb to curb.

Standard Plan A-40.20, Detail 3 or Detail 4 shows HMA ¼ inch higher than concrete. This should apply to all paving up against any hard materials in the surfacing, such as steel joints or headers, for the following reasons:

1. This insures compaction effort is applied to the HMA and not the hard material. Lack of compaction in the butt joint is the primary reason for raveling and early failure of HMA, which is a chronic maintenance problem. It is acceptable for HMA to be placed flush at the gutter line to avoid ponding where compaction is not critical.
2. The slightly raised grade prevents snowplows from destroying the bridge joint.
3. Within a short period of time, the tires provide additional compaction and/or rutting that will produce a smooth surface with the best performance.

6-10 Concrete Barrier

SS 6-10.3 Construction Requirements

SS 6-10.3(2) Cast-In-Place Concrete Barrier

On some projects, the Contractor has the option of using slipform techniques in addition to the usual fixed forms as specified in *Standard Specifications* Sections 6-02.3(6), 6-02.3(11)A, 6-02.3(24)C, 6-10.3(2), and 9-03.1(2)B.

In either method, barriers and rail bases should be carefully aligned both horizontally and vertically to give a pleasing appearance; refer to *Standard Specifications* Section 6-01.4. The vertical adjustment for the pleasing appearance is intended for localized camber and deck profile variables. This adjustment is not intended to eliminate grade breaks, such as vertical curves and superelevation transitions. The Project Engineer should plot to a large scale the profiles of the roadway grades at the curb lines. From these profiles, the grades for the tops of traffic barriers, pedestrian barriers, and rail bases can be properly determined. A slight hump in the barriers or rail base over the whole bridge is not usually objectionable.

On the safety-shape traffic barriers, some of the height variation may be accommodated in the vertical face at the base. Any height variation shall maintain the 2 foot 8 inch total height. The vertical toe face at the base is usually 3 inches unless the structure is receiving an immediate overlay. To accommodate the overlay, the vertical face at the base is increased to 3 inches plus overlay thickness. The front face geometry of the safety-shape traffic barrier is critical and should not be varied except as noted herein. Ideally, all height adjustment required to provide a pleasing appearance should be accomplished by modifying the total height of the traffic barrier by varying the vertical toe face at the base, i.e., 2 inch minimum. The front and back faces of the traffic barrier are parallel on the upper part to accommodate all height adjustment necessary. The 7 inch height of the intermediate sloping face shall be maintained. To ensure proper alignment, carefully check the top of forms or the Contractor's control wire prior to placing concrete.

On slipformed traffic barriers and pedestrian barriers, the same cross-Section as shown for fixed-form construction shall be used, except the top chamfer may be shaped to a ¾ inch radius. Although slipforming may be allowed in the contract, the reinforcing steel bars may not be sufficient to resist the forces during the concrete placement operations. The contractor should evaluate the stiffness of the reinforcing and, if necessary, provide additional reinforcing steel crossbracing, both longitudinally and transversely. Slipformed concrete is usually placed with a slump of 1¼ inches plus or minus ¼ inches. This slump is critical and should be carefully controlled by the Contractor. It is not unusual to encounter conditions which produce sections of unsatisfactory barrier or rail base due to slump, finish, alignment or other problems. When this occurs, do not hesitate to have the unsatisfactory sections removed. Occasional removal is inherent in slipform construction.

Placement of the reinforcing steel bar cage to ensure adequate concrete cover and proper reinforcing bar location is very important and difficult to check for slipformed traffic barrier, pedestrian barrier, and rail bases. When fixed forms are used, final adjustment of the reinforcing steel bar cage can be accomplished after the forms are set prior to concrete placement. The slipform method does not present this opportunity. For that reason, *Standard Specifications* Section 6-02.3(24)C requires that the Contractor check reinforcing steel bar clearances and placement prior to slipform concrete placement. This check can be accomplished by either the use of a template or by operating the slipform machine over the entire length of the barrier. The final grade control must be set prior to the check. All reinforcing steel deficiencies must be corrected by the Contractor.

SS 6-10.3(5) Temporary Barrier

The condition of temporary concrete barrier shall be verified with a visual inspection by the Engineer. Any section of temporary barrier determined to be in good condition is allowed to be used on the project. Any section of temporary barrier determined not to be in good condition shall be handled as follows:

1. For temporary barrier sections being placed in a new run of temporary barrier: Any section(s) deemed not to be in good condition by the Engineer will be rejected and are not allowed to be installed in the new run of temporary barrier. The rejected barrier section(s) shall be removed from the project.
2. For temporary barrier sections that have already been placed in a run of temporary barrier: Any section(s) which are deemed not be in good condition by the Engineer shall either be repaired immediately to the Engineer's satisfaction, or the section shall be removed from the temporary barrier run and replaced with a section of temporary barrier determined to be in good condition by the Engineer. The rejected barrier section(s) shall be removed from the project.

Temporary concrete barrier sections shall be deemed to be in good condition and may be accepted when they have:

- Only minor blemishes (i.e. dirt, scuffs, traffic marks, superficial surface cracking, etc.)
- No excessive amounts of cracks (1/2 inch or deeper) or chips
- No spalls in the concrete with a depth greater than 1.5 inches
- End connection hardware that is intact, undamaged, and functional

Temporary concrete barrier sections shall be deemed not to be in good condition and rejected when they have:

- One or more cracks that penetrate through the entire section
- One or more spalls in the concrete with a depth of greater than 1.5 inches
- Exposed rebar or bolts that are protruding through the barrier surface
- Cracked or broken concrete that could be easily dislodged if struck by a vehicle
- End connection hardware that is deformed, bent, broken, corroded/rusted, or no longer functional

6-14 Geosynthetic Retaining Walls

GEN 6-14 Description

Geosynthetic retaining walls may be Standard Plan walls or specially designed walls that are used in both permanent and temporary applications. Permanent walls usually have different material acceptance requirements than temporary walls and usually have a facing to protect the geosynthetic from damage and sunlight. In temporary applications, it is common to see a Standard Plan wall called out in the Plans. When this occurs, the wall is still a temporary wall. The Standard Plan wall was called out because the internal design of the wall has already been completed for the Standard Plans. This simplifies the submittal process and speeds up construction, as internal design is not needed.

Regardless of the wall's status as permanent or temporary, most geosynthetic walls require the Contractor to do some geometric design to lay out wall lift heights and layer elevations to meet the specific geometry needs in the Plans and achieve the proper grades and lines of the Contract.

SS 6-14.2 Material

Geosynthetic reinforcement for permanent geosynthetic retaining walls are accepted on receipt of "Satisfactory" test reports from the State Materials Laboratory. Sampling must be completed by a tester qualified in sampling geosynthetic material.

Section 9-33.4(3) defines a "lot" and outlines the process for retesting. The Project Inspector must be familiar with the retesting procedures and understand the definition of a "lot". It is important to discuss the acceptance procedures with the Contractor well before the material is needed, as testing can take up to 30 days.

Geosynthetic materials for temporary geosynthetic walls do not need to be tested and are accepted by Manufacturers Certification of Compliance, unless specified otherwise in the Contract. The handling and storage requirements in Section 3-09.2 apply to both permanent and temporary geosynthetic retaining wall materials.

Gravel borrow for structural earth walls is used in the construction of geosynthetic retaining walls. Refer to section SS 3-03.3(13), Borrow for more information.

SS 6-14.3 Construction Requirements

Type 2 Working Drawings must be followed by the Contractor with respect to geosynthetic material type, material strength, and geosynthetic reinforcement length. The Project Inspector must ensure the requirements for backfill and compaction are met. Temporary geosynthetic retaining walls have the same construction requirements as permanent retaining walls.

SS 6-14 3(2) Submittals

The Contractor is required to submit Type 2E Working Drawings complying with the requirements of the *Standard Specifications*, Standard Plans, and Contract Plans prior to Work performed on the geosynthetic retaining wall. The Project Office should verify that working drawings include all required submittal elements and have the correct plan and profile geometry prior to forwarding for further review per Figure 1-1 (Working Drawings, Shop Plans, or Submittal Type).

Geosynthetic retaining wall designs are provided in the Standard Plans and/or Contract Plans for each Contract. The designs dictate the wall geometry, material strength requirements, and geosynthetic reinforcement length. The Contractor, by way of the Type 2E Working Drawings, can determine where steps are needed to facilitate the plan profiles. The Contractor also can determine the lift thicknesses for each layer, as allowed by the wall design. The geosynthetic reinforcement lengths shown in the Contract documents are based on wall height and do not change. This is an important note when determining measurement limits.

SS 6-14.4 Measurement

The Standard Plans or Contract Plans show the measurement limits for structure excavation, backfill, and compaction as the limits of the geosynthetic reinforcement. At abutment walls, measurement for the face of wall would include both the area of the wall parallel to the roadway as well as the area of wall transverse to the roadway. In that same situation backfill is only measured once.

If wall drains are called for in the Plans at the back of the reinforcement, the measurement for backfill should be extended to the back of the drains. As stated previously the reinforcement lengths are determined by the design provided in the Contract and do not change.

In a cut wall situation, it may then be necessary to compensate the Contractor for required excavation beyond the limits of the reinforcement under the Specifications for shoring or extra excavation class A or shoring or extra excavation class B, per the Contract documents. In either case, backfill for extra excavation and shoring is included in the shoring or extra excavation items. Based on Section 3-03.4 embankment compaction is also measured.

6-19 Shafts

GEN 6-19 Shafts

Drilled shaft construction is technical it is critical to follow established guidelines, practices, and procedures to avoid costly fixes and cause safety issues to the public. Drilled shaft foundations require close attention to details during construction.

Training on drill shaft construction is available through the State Construction Office. The training covers specifications, equipment, site geological conditions, and other general topics.

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 Project 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 Project Engineer, an additional conference shall be held before additional shaft construction operations are performed.

Nondestructive QA Testing of Shafts

There are two main types of non-destructive tests that WSDOT allows for drilled shafts. Cross-hole Sonic Logging (CSL) and Thermal Integrity Profiling (TIP). Either method is acceptable. Both tests need to be performed by an experienced tester, and the findings/ report require the seal of the engineer in responsible charge of the testing. Shafts poured in the dry do not require nondestructive testing, but all others do.

For Quality Assurance (QA) purposes, WSDOT has moved to contractor supplied non-destructive testing of shafts. This means that a testing subcontractor will test the shafts and provide the Contractor with the report. The Contractor is then responsible to forward the report to the Project Engineer.

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.

The Project Engineer is responsible to review the report and if the test report does not identify anomalies, the Project Engineer may allow construction of the shaft to continue. If the test report identifies anomalies, the Project Engineer shall not allow shaft construction to continue on that shaft. The Project Engineer may also suspend further shaft construction on the project. The Project Engineer shall forward the test results and Inspector Daily Reports to the Construction Office. The Construction Office will provide the Project Engineer with further instructions.

WSDOT retains the right to perform Quality Verification (QV) testing on 10 percent of the shafts that are tested by the Contractor. The purpose of the QV testing is to verify the Contractor's results (currently, WSDOT is only performing CSL tests for verification). The Project Engineer is responsible to select shafts for QV testing and shall coordinate the testing with the Contractor and the Geotechnical Office. Ideally, WSDOT QV tests should occur on the same day as the Contractor's QA test, to minimize delays and impact to the Contractor. If the Contractor has selected TIP testing for shafts, the PE must identify the QV shafts during cage fabrication so that CSL tubes can be installed when the cage is fabricated. Once the shaft concrete has been successfully placed, call the Geotechnical Office to schedule QV CSL testing.

Contact	Name	Phone	Email
Primary	Bob Grandorff	360-709-5468	GRANDOR@wsdot.wa.gov
Alternate	Ioanna Kladou	206-440-4271	KLADOUI@wsdot.wa.gov
Alternate	Mark Frye	360-951-7267	FRYEM@wsdot.wa.gov
Alternate	Andrew Fiske	360-709-5456	FISKEA@wsdot.wa.gov

A key aspect of the QV test is comparing the WSDOT test results to the Contractor's test results. Accordingly, the PE will need to forward the Contractor's test report to the Geotechnical Office as soon as possible. The Geotechnical Office will prepare a report for the QV test. If the WSDOT test corroborates the Contractor's test, no further action will be necessary. If the tests disagree, the Construction Office will provide the Project Engineer with further instructions.

6-20 Buried Structures

6-20.1 Construction Requirements

Many of the instructions for the construction of culverts covered in Section 7-02 are equally applicable to the construction of buried structures.

Equipment should not operate across buried structures until the backfill has been constructed as required in the Installation Plan (see *Standard Specification* Section 6-20.3(2)E).

Precast Concrete Structures

Fabrication

Precast forms may not be able to produce the exact geometry shown in the Plans. *Standard Specification* Section 6-20.3(3)A allows for alternate designs to accommodate minor deviations. When the deviations exceed the limits specified, the alternate structure proposal must meet the Value Engineering Change Proposal (VECP) requirements in *Standard Specification* Section 1-04.4(2)A.

For Class 1 and Class 2 precast concrete three sided and split box structures, unless otherwise shown in the Plans, the Contractor is required to progressively shop assemble the top and bottom units of at least the first three adjacent segments for inspection of fit-up. The shop assembly may be considered successful if:

- Units and segments align and meet construction tolerances
- Point or edge loading does not occur within joint locations

The fit-up should be performed at the fabrication plant on a flat level, solid surface such as concrete or solid ground with a layer of sand. Bunking or shimming shall not be allowed during the shop assembly.

- WSDOT Fabrication Inspectors are typically onsite to observe the fit-up and verify tolerances are met. It is encouraged that a Contractor's representative and staff from the Project Office also attend to observe. As an alternate to being physically present, the Contractor and Project Engineer may agree to observe the fit-up via a video conference. The Project Engineer may delegate a representative to accept the fit-up. The Fabrication Inspector should not stamp Approved for Shipment until the Project Engineer or delegate has approved the fit-up. This should all be discussed at the Pre-Construction Conference.

Bedding Preparation

The quality of the bedding prepared by the Contractor has significant impact on the difficulty of installation and quality of the final project. It is best practice to install bedding for precast concrete structures as flat and level as possible on the required grade.

High or low points in the bedding or soft/unsuitable materials can cause gaps or steps in the joints between segments. A mud slab or leveling pad may be considered as bedding preparation for some precast concrete structures; it could provide a flat, even, dry, and firm foundation and prevent loose material from inhibiting jointing efforts.

Installation Plan

Standard Specification 6-20.3(2)E requires the Contractor to submit a detailed Installation Plan. It is important that this Installation Plan address in detail the circumstances described below. Sequence of installation is important. The installation plan should be such that precast units can be lowered into their final position as much as possible. It is best practice to avoid sliding units on bedding material, as loose or disturbed bedding material can become trapped between the segments. When sliding the units cannot be avoided, trapping material within the joint can be prevented by preparing a narrow channel across the front of the previously set segment to capture any loose or disturbed material as units are jointed. Alternatively, a metal plate, or other material, could be placed over the bedding material between adjacent segments.

Precast segments may need to be pulled together to achieve the required joint openings and maintain compression of the butyl rubber sealant. Once the required joint gap is achieved, jointing forces should be maintained for at least 30 seconds to ensure the butyl rubber gasket is seated and distributed evenly within the joint. It may be necessary to brace or weigh down previously set segments to inhibit movement during jointing.

Ensure precast concrete segments join squarely. Also, it is best to avoid jointing one side and then trying to join the other side, as this can cause damage to the segments.

Bedding material leveling adjustments can be made after unit placement by a gentle side to side movement of the unit to encourage it to settle into the bedding.

The Contractor may make alignment adjustments by jacking from the side of the trench or trench box, ensuring the jacking load is distributed over a large area on the precast segment to avoid damage. They can also be made by gently pulling or pushing with an excavator (using a suitable buffer).

Leveling and alignment adjustments should not be done using a downward pressure on the segments.

It is important to maintain geometric control and perform real time geometric checks as units are placed. This will help avoid:

- Over or under runs for the total length of the precast concrete structure
- Misaligned vertical joints between lower and upper
- Out of tolerance joint adjustments

Point loadings on precast concrete structures should be avoided. Point loadings can be caused by construction equipment, uneven joints or lack of adherence to tolerances. They can cause corner, edge or "dinner plate" spalling or shear cracks.

If the Installation Plan has not addressed some of these topics, they should be discussed at the Pre-Construction Conference so there is no misstep while the Work is being performed.

Structural Plate Structures

Assembling

Those inspecting the installation of metal plate structures should be familiar with the requirements of AASHTO LRFD Bridge Construction Specifications Section 26. *Standard Specifications* Section 6-20.3(8)B requires construction of metal plate structures to conform to these requirements.

All Class 2 structural plate buried structures shall meet the structure dimension tolerances for the assembly of long span structures. Structure shape shall be checked regularly during construction by the Contractor as described in *Standard Specification* Section 6-20.3(9)A. Installation deflection inspections by direct measurement shall be performed by the Contractor immediately after construction and 30 days or more after construction as described in *Standard Specifications* Section 6-20.3(9)A. These activities are critical as metal structural plate structures may not function or distribute loads properly otherwise.

Manufacturers of multi-plate structures are required to supply detailed assembly instructions with their structures, which should be closely followed.

Plates on different parts of the structure can be of different sizes and thicknesses. Ensure that the correct plates are used in the correct locations. Plates should be labeled with unit identifiers shown in the working drawings as well as the thickness or gage in accordance with *Standard Specifications* Section 6-20.3(7)B.

It is important that the bottom plates be correctly positioned for alignment and grade of their edges before the other plates of the section are bolted up so the completed structure will be in proper alignment. If the structure starts to creep or spiral, the only way to correct this condition is to remove the plates to where it is in correct alignment and reconstruct the structure.

High-strength bolts are used in bolting the plates together. In order for the connections to function as designed, the bolts must be tightened to the specified tension. Section 6-03.3(33) covers the instructions for construction and inspection of high tensile strength bolts. Impact wrenches must be calibrated as specified since overtightening may overstress the bolts and under-tightening will not give the connection the required strength. If more than one crew is assembling the structure, the impact wrenches must be calibrated to tighten the bolts to the same torque. Bolts have been observed to loosen and back out during compaction of backfill so it may be beneficial to tighten the bolts towards the higher end of the range of recommended torque.

Submittal

For Contractor supplied designs, the Project Office shall ensure fabrication shop drawings are not submitted prior to approval of the site specific Plans, Specifications and supporting calculations. Fabrication shop drawings shall reflect any and all comments made during the review of the Plans, Specifications and calculations.

To help reduce the duration and number of review cycles, it is recommended that WSDOT reviewers for buried structures be available for direct meetings with fabricators to help resolve review comments. This will help address concerns raised by precasters that the duration and number of review cycles is causing hardship during construction.

Recommended meeting invitees include the Project Engineer or designated representative, ASCE, BTA, Contractor representative, and fabricator. The BTA or Bridge and Structures Office reviewer shall ensure all communication conforms to *Construction Manual* Section GEN 1-00.11.2.