5-15.1 Preliminary Review

At the first opportunity, the engineer and inspector should make a thorough study of the plans, special provisions, and sections of the standard specs relating to the work under the contract. The structure plan's general notes, bid items, quantities, dimensions, and elevations should also be reviewed. The most common error is in the number and length of reinforcing bars. The inspector should check for conformance between dimensions given and elevations shown for the various structure elements.

The structure site must be inspected to determine if conditions are as depicted on the plans or whether changes have taken place since the original survey was made. This investigation may indicate the structure should be shifted slightly from the exact stationing shown on the plans to better fit existing conditions. There may be no latitude for a shift in position in the case of a grade separation structure. However, there is sometimes opportunity to shift drainage structures. The Bureau of Structures should be consulted before making any changes from plan location, skew, elevation, grade, etc.

5-15.2 Structure Construction Survey

The centerline or reference line must be rerun and referenced at convenient locations beyond the limits of the structure where the references will be available throughout the construction operations. The location of each substructure unit should be staked in the manner described under CMM 7-40, and a stakeout diagram prepared, showing the location and relationship of all stakes set, and the checks made to verify their accuracy. When staking an abutment, it may be convenient to stake the centerline of bearing, or front or back face, depending on which line the unit is dimensioned from on the plans. In any case, the contractor must be informed which line has been staked. When applicable, the horizontal and vertical clearances should be checked.

The staking and layout of complex structures in complicated interchanges require special procedures adapted to the particular requirements. When grading operations are in progress, staking procedures demand close liaison with the contractor to maintain and preserve essential points.

The engineer is responsible for the setting and accuracy of stakes or reference points needed to establish the location and alignment of substructure units, and for establishing one or more convenient bench marks in the immediate vicinity of the work from which elevations can be determined. The contractor is responsible for all additional stakes, batter boards, markings, etc., needed to facilitate layout or construction of the work, and for the final conformance of the structure with the lines, grades and dimensions shown on the plans. The engineer can cooperate with the contractor in establishing the additional lines and elevations necessary for the proper prosecution of the work. However, it should be clearly understood that in so doing WisDOT is not relieving the contractor of any of the contractor's responsibility for the lines and grades.

5-15.3 Excavation for Structures

Excavation for structures includes all excavation, except roadway and drainage excavation and excavation relating to structure removal, necessary for placement of the structure such as excavation for foundations, girders, projections, timber backing, tie rods, dead-men seals, and sub-foundation courses.

Standard spec 206 provides that the item of excavation for structure will be measured as a unit and paid for as a lump sum and that the lump sum bid price applies to excavation removed to an elevation between planes lying one foot above and below the elevation of the bottom of footings or floor of culverts, or the invert of structural plate pipe or pipe arches as given on the plans for the specific units. Excavation ordered to be performed to another elevation either above or below the planes described above, in order to obtain satisfactory foundation conditions, will be paid for as extra work as provided in standard spec 104.2.2.1.

Backfilling, including sub-foundation course, is incidental to the item of Structure Excavation. It is essential that an agreement be reached with the contractor as to the basis of payment for excavation to be paid as extra work. The actual quantity involved in the extra work may be more or less than the amount involved in the original lump sum price, depending on whether the footing was raised or lowered from the original plan elevation. A contract change order formalizing the agreement should be promptly executed.

All necessary clearing and grubbing will be measured and paid for separately. Structure backfill, when specified, will be measured and paid for separately.

Although the structure excavation is on a lump sum basis, it is recommended that adequate elevations or cross-sections of the site be obtained before excavation begins. Elevations or cross-sections will be of value should a change order need to be negotiated in connection with extra work.

The excavation should be carried to the elevation of the bottom of the footing shown on the plan. A possible exception would be when a satisfactory foundation in rock can be secured at a higher elevation.

The material encountered at the plan elevation of the footing should be compared with that indicated by the boring information shown on the plans. If the foundation conditions are not equivalent to that indicated by the borings, it could be necessary to revise the footing elevation, increase the size of the footings, or extensively redesign the substructure unit.

Variations in foundation material are not as critical for foundations supported on piling as in the case of spread footing designs. Subsurface conditions are quite variable in most of Wisconsin, and change so rapidly that boring results only a few feet away may be completely unreliable, so caution is advised when dealing with structure foundations. The region soils engineer should be consulted when soil conditions differ significantly from plan. Do not place concrete on frozen subgrade.

If satisfactory material is not obtained at the plan elevation, the nature and extent of the material below that level should first be determined. This can usually be done by hand boring, rod sounding, or digging a test hole, but in extreme cases, arrangements can be made to have one of the department's boring rigs brought in. If satisfactory foundation material is found at a reasonable depth below plan elevation, lowering the footings will probably be the best solution; if not, a redesign may be required. The Bureau of Structures, Structures Design section should authorize all but minor changes from plan elevations in advance.

When foundation piling is to be driven in a footing, the excavation must be completed to plan elevation or slightly below, to allow for swell of the material displaced by the piling, before driving of the piling begins. After pile driving has been completed, any displaced material must be removed to plan elevation of the footings. Pile driving is discussed in detail in CMM 5-40.

Structure excavation material may be disposed of in accordance with standard spec 206.3.14 as riprap, backfill, or embankment construction if requirements are met.

5-15.4 Forms and Falsework

5-15.4.1 Forms

Forms must be designed to withstand the fluid pressure of concrete that has not reached initial set, plus a live load allowance. This design depends upon the rate of pour and the temperature of the concrete, and dictates the spacing of the form ties and braces. The formwork plan may be requested by the engineer for review, and must be revised by the contractor if so ordered by the engineer. Review of the plans does not relieve the contractor of responsibility for obtaining satisfactory results as specified in standard spec 502.3.3.1.

The contractor may perform prefabrication of formwork and reinforcing steel assemblies at their own yard or shop site for some components of the structure. This can result in improved quality of the structure because the fabrication occurs under more favorable controlled conditions at the shop. If the contractor intends to utilize formwork/rebar assemblies prefabricated off the project site, the engineer must be notified in advance of the start of the prefabrication work. This will allow the engineer the opportunity to provide whatever level of inspection appropriate for the off-site prefabrication work. The contractor must keep on file all required certifications for materials incorporated into the prefabricated assemblies, including heat number tags from the reinforcing steel.

If the contractor plans on storing prefabricated formwork/rebar assemblies for an extended period of time before incorporation into the work on the structure, it is recommended that the assemblies be protected from repeated rainfalls. This will avoid potential problems with:

- Distortion for the formwork assemblies.
- Excessive rusting or pitting of reinforcing steel.
- Washing out of the form oil from the assembly, which cannot be replaced because of tight clearance with reinforcing steel.

Regardless of whether the initial fabrication of these assemblies is inspected at the contractor’s shop, the engineer reserves the right to final acceptance of these assemblies at the project site at the time of incorporation into the structure based upon the condition of the prefabricated assembly at that time.

The engineer must inspect the forms before the start of placing concrete for correct dimensions and alignment, and require correction of any obvious inadequacy. Checking of the formwork should continue as the work proceeds. Forms should be observed throughout the progress of the pour to detect any excessive deflections or apparent weakness. Despite checks, the contractor assumes the full responsibility for the adequacy of the formwork plan, adequacy of form ties and braces, and for obtaining satisfactory results that conform to the specified lines and dimensions.

Forms are to be built mortar-tight. Wood forms are sometimes built in panels or sections as an early operation on the job, for assembly and use at a later date. Shrinkage of the lumber may cause objectionable openings of the joints between boards. A liberal application of form oil, and occasional wetting with water, will minimize shrinkage.

Steel forms used for pier columns should be carefully inspected to see that individual sections line up properly, present a smooth surface, and are mortar-tight. Forms with open joints that will allow mortar to escape or that will result in fins or ridges in the finished work cannot be used.

When fiber pulpboard tubes are permitted as forms for cylindrical columns, it is customary for the contractor to order the forms to the lengths shown on the plans and have them delivered before the excavation is performed and the footings poured. If it becomes necessary in the course of the excavation to lower the footings, the column forms will be too short. Standard spec 206.3.2 provides that the elevation of footings shown on the plans are approximate only, and the engineer may order changes in dimensions and elevations of footings as may be necessary; so, strictly speaking, the contractor should be required to furnish another form of the revised length, since it is impractical to splice this type of material. In most cases, however, it will be satisfactory to thicken the footing to restore the top to the original elevation, or construct a square pedestal atop the regular depth footing, so the original length of form may be used. The volume of concrete required to construct the additional length of the cylindrical column corresponding to the amount the footing was lowered will be paid for as Concrete Masonry, but any additional volume required to thicken the footing outside the limits of the column will be the contractor's responsibility.

5-15.4.2 Falsework

Falsework consists of temporary supports required for the construction of the permanent structures. Falsework may be required for the support of structural steel during erection. This discussion will apply primarily to the support of concrete pier caps, slab span bridge floors, etc., while the concrete is being placed and cured.

As in the case of forms, the contractor is responsible for obtaining satisfactory results with the falsework used. However, many instances may be cited where the design and construction of falsework was inadequate, and although there was no complete failure, excessive settlement or deflections resulted in the appearance or riding quality of the completed structure being somewhat less than satisfactory. To avoid such deficiencies, one copy of the falsework plans is to be submitted by the contractor to the engineer for review. Up to two additional copies may be requested. The contractor must make changes required by the engineer. Submittal of the plan or concurrence in the plan by the engineer does not relieve the contractor of full responsibility of adequacy of the falsework. Review of the plans does not relieve the contractor of responsibility for obtaining satisfactory results as specified in standard spec 502.3.2.

The contractor should submit only plans that are clear and legible. The plans should show:

1. A layout of the falsework location and spacing of falsework bents.
2. An elevation view of the falsework bents, showing spacing of piling or columns, caps, and sway bracing.
3. Minimum bearing for piling, which is usually in the range of 20 to 40 kip.
4. Presumed allowable soil pressure if mudsills are used.
5. Cross-section of deck forming.
6. Size and spacing of all lumber and timber. All lumber will be assumed to have four sides surfaced unless otherwise shown.
7. Continuity of members over two or more spans, if continuity is to be considered in the analysis of the falsework.
8. The species of wood and its stress grade. In the absence of specific information, it will be assumed the material to be used is equal to or better than Pacific Coast Douglas Fir, No. 1.

   Allowable unit stresses that permit 25% overstress due to short-term loading in falsework are 1,870 psi in bending, 145 psi in horizontal shear and 1,490 psi in compression parallel to the grain and 490 psi in compression perpendicular to the grain. The modulus of elasticity is 1.76×10^6 psi.
9. Shim arrangement proposed to achieve the crown. Bearing surfaces should be at least 2" wide for proper support.
10. The plans must be signed and sealed by a registered professional engineer.

Falsework plans should not be sent to the design section of the Bureau of Structures except for unusual situations.

A common cause of excessive settlement of falsework is yielding of the supporting foundations. Falsework for substructure elements, where a slight amount of settlement may not be critical, is often supported by posts and beams or scaffolding resting on the footings or on timber "mudsills" placed directly on the ground. Piling should generally support falsework for superstructures, unless the foundation material for the support of timber mudsills is extremely firm. Falsework columns should not be placed on spread footings unless the footings are supported on piling or founded on rock. The region should not accept falsework plans from the contractor proposing the use of timber mudsills, except under the best of conditions permissible under the specifications, and in addition, should be very cautious about permitting the use of any timber mudsills on embankment benches. Standard spec 502.3.2.3 permits falsework to be supported by timber mudsills placed on paved, well compacted slopes of berm fills; however, the falsework cannot be strutted to the pier columns unless the columns are founded on rock or supported by piling.

The structure plans usually contain a deflection diagram indicating the amount of camber that must be built into the bridge floor to compensate for the dead load deflection that takes place when the falsework is released, plus an allowance for future creep. The falsework and forms must be built to provide this required camber after the concrete is placed, which means that a further allowance must be made for the settlement that will take place in the falsework itself. The amount of settlement that will occur is not subject to precise determination, and will depend on the character of the foundation support and the amount of compression occurring at the timber joints or interfaces in the falsework. The latter is sometimes estimated on the basis of from 0.03" to 0.06" per joint where the stress is perpendicular to the grain of the wood. Falsework plans must be designed for anticipated deflections of individual falsework members as well as for allowable unit stresses. Excessive deflections that are detrimental to the ride and appearance of the structure are a common deficiency of falsework designs.

When falsework is built over a stream or lake used for boating, adequate horizontal and vertical clearance should be provided for the passage of rowboats and small powerboats.

The minimum vertical clearance over a highway or street used by traffic is 13.5', and the minimum horizontal clearance is 22', unless otherwise provided by the plans or special provisions.

5-15.4.3 Failures

Frequently when forms or falsework fail to perform properly during the placing of the concrete, the cause may be one or more of the following:

1. Formwork design weakness is commonly in details rather than in the main structural members, unless the members are of poor quality, since designs similar to those involved in failures have been successfully used on other occasions.
2. Details that are difficult to perform will not be properly performed and may start a failure. These items should be eliminated in the planning stage.
3. High shoring becomes particularly susceptible to failure when not adequately braced diagonally.
4. Shock and vibration may result from the use of duckboard runways.
5. Power buggies, in synchronism at high speeds, impose a lateral force, which must be provided for in the design and details.
6. Forms are continuously supported structures and must be provided with uniform bearing at each support. Otherwise, settled mudsills or shrinkage at timber post splices will completely upset the computed reactions with possible overloading of some posts.
7. Wedging of posts to counteract compression under load must be done under proper supervision so a previously properly assembled form support is not disrupted.

5-15.5 Reinforcement

5-15.5.1 Sizes and Grades

Bar steel reinforcement is basically round in cross-section with the surface deformed (ridged) to improve the bond with the concrete. In the U.S. Standard System of Measurement the size of the bar is designated by a number that indicates the diameter of the bar in eighths of an inch. Thus, a No. 4 bar is 4-eighths or ½-inch in diameter, and a No. 8 bar is 8-eighths or 1-inch in diameter. The exact correlation holds true in sizes up to No. 8, but deviates slightly in larger size bars.

Often, bars delivered to a project are stamped with the metric designation number. The metric number is the bar diameter rounded to the nearest millimeter. Table 1 shows the relationships between U.S. Standard and metric bar sizes.

Table 1 U.S. Standard and Metric Bar Sizes

U.S. Standard Bar No.	Diameter		Metric Designation Number
	in.	mm	
3	0.375	9.5	10
4	0.500	12.7	13
5	0.625	15.9	16
6	0.750	19.1	19
7	0.875	22.2	22
8	1.000	25.4	25
9	1.128	28.7	29
10	1.270	32.3	32
11	1.410	35.8	36
14	1.693	43.0	43
18	2.257	57.3	57

Reinforcing steel is also classified by grade. Grade 40 must achieve minimum yield strength of 40,000 psi. Grade 60 must achieve minimum yield strength of 60,000 psi. Grade 40 deformed bars have no specific grade identification marks on the bars. Grade 60 deformed bars may be identified by the number "60" or by the presence of a single continuous longitudinal line offset from the center of the bar side and extending through at least five deformation spaces.

5-15.5.2 Storage

Standard spec 505.3.1 states that reinforcement must be stored above the ground on platforms, skids, or other supports, and must be protected so far as practicable from mechanical injury and deterioration caused by exposure. When placed in the work, the reinforcement must be free from detrimental dirt, dust, paint, oil, or other foreign material.

Generally, new reinforcing steel can be stored on supports above the ground, without protective covering, for periods of from one to three months. When reinforcing steel is stored for longer periods, it should be protected from the weather with adequate covering. Refer to the Coated Bar Steel Reinforcement section for specific storage requirements for coated rebar.

The covering should be placed to allow air to circulate to the steel from beneath. Storage should be, whenever possible, in well-drained, stable areas not likely to become soft from rains, snowmelt, or freezing and thawing. If these conditions are likely to be encountered, extra blocking should be used to avoid shifting.

5-15.5.3 Rust

Test data has indicated that deformations, rather than surface conditions, are the principal parameter determining the bond characteristics of reinforcing bars. Therefore, tight light rust and thin powdery rust is not considered detrimental and need not be removed.

When reinforcement is rusted to the extent the effective cross-sectional area appears to be reduced, evidenced by an overall coating of thick rust or heavy scaling, tests should be made to determine if the reinforcement conforms to the required dimensions and mechanical properties. If the reinforcing steel meets these requirements, the rust should not be a cause for rejection.

When the condition of the steel is such that removal of rust is desirable, consideration should be given to the fact that, frequently, normal handling before installation is sufficient for the removal of loose rust and scale that might otherwise impair the bond of the steel with the concrete.

"White rust" frequently occurs on zinc coated steel members closely stored or nested together. If it is merely a powder easily rubbed or scraped off, and if the underlying steel is not scaled or has not lost cross-sectional area, the members can be accepted and used. If there is significant loss of section, proceed as noted above for rust. Avoid "white rust" altogether by allowing sufficient air to flow through stored zinc coated steel.

5-15.5.4 Coated Bar Steel Reinforcement

Present design practice calls for coated high-strength bar steel reinforcement to be used in all layers of reinforcement in bridge decks and parapets. The steel must be totally coated, including the ends. It is essential to handle coated reinforcement very carefully before and during installation. Standard spec 505.3.1 requires using padded or nonmetallic slings and padded straps when handling the reinforcement.

Before placement, damaged areas must be repaired with the manufacturer-supplied patching or repair material before the start of rusting or contamination. Before the pour the epoxy coated bars should be re-inspected for damage that may have occurred during placement. Ties must be an approved plastic or nonmetallic material, or be metal wire coated with nylon, epoxy, or plastic.

Coated bars should be stored on protective timbers with the supports spaced close enough to prevent sags in the bundles. If the storage of epoxy-coated bars is expected to last longer than two months, the bars must be protected from sunlight with opaque polyethylene sheeting or other suitable protective material. The contractor should ensure that the covering is adequately secured, while also providing for air circulation to minimize condensation. If epoxy-coated bars have been placed into a bridge deck mat and the deck pour is not to proceed for two months or more, the contractor must cover the rebar to protect from ultraviolet radiation.

Epoxy-coated reinforcement must not be welded or flame cut.

5-15.5.5 Field Cutting of Reinforcing Bars

Uncoated bars can be cut with a saw or flame-cut (such as with an oxy-acetylene torch). Epoxy coated bars can be cut with a saw, but cannot be flame-cut.

5-15.5.6 Testing and Acceptance

Before reinforcement is incorporated in the work, evidence of acceptance by test, or as otherwise permitted for small quantities, must be in the hands of the engineer. A Certificate of Compliance is required for coated high-strength bar steel reinforcement.

For each size of bar that exceeds 50,000 pounds on the project, a 5-foot cut sample piece must be provided to the engineer for testing. Refer to CMM 8.50 for details of the required samples. When replacing the cut sample piece it's important that the contractor supplies the original 5' length plus additional lap length required for the splice. Splice lengths typically vary between 30 - 60 bar diameters. An old rule of thumb for splice lengths in uncoated rebar is to provide 38 times the rebar diameter. However, the WisDOT required splice length is a function of concrete strength, reinforcing yield strength, concrete cover over the bars, whether any horizontal bars have concrete below it more than 12", rebar diameter, and whether the bars are epoxy coated.

The WisDOT LRFD Bridge Manual provides information about development lengths for splices in section 9.3.1, and the splice lengths are provided in Table 9.9-1 and 9.9-2 of the bridge manual.

5-15.5.7 Placement and Fastening

Reinforcing bars must be placed and rigidly held in the correct position if they are to serve their intended purpose. The inspector should check on the size and spacing of the bars, as they are being set, the manner in which they are being supported, and the clear distance from the form or surface, as well as the correctness of any bends specified. Clear distance between forms and stirrups may vary by 1/2" due to the allowable tolerance in fabrication of stirrups. The inspector should also ensure the steel reinforcement when placed in the positions shown on the plan is firmly held in those positions to prevent movement during or after the placement of concrete surrounding the bar. Standard spec 505.3.4 provides for placing and fastening steel reinforcement. The wire used for the tying down of the steel must be of sufficient strength to prevent breaking or stretching of the wire during pouring operations. Standard zinc coated 0.15" diameter wire is recommended for uncoated steel bars. A single loop of standard tie wire is not sufficient. Plastic coated wire or alternates listed in standard spec 505.3.4 must be used to tie down coated bar steel.

Standard spec 505.3.4 provides that bars must be securely tied at all intersections, except where spacing is less than one foot in each direction, in which case, alternate intersections must be tied. The correct application of this spec requires that bar spacing must be less than one foot in both directions before tying of alternate intersections is permitted. The inspector should record the quantity, size, and length of all bars placed in each unit of the structure.

The engineer should not measure and pay for the extra rebars used to accommodate for provided girders that have more camber than what the plans indicate.

Spot welding of reinforcing bars to each other or to various devices used in holding forms in place will not be permitted. Bar mats or cages, when fabricated, should be tied. Welding or mechanical butt splicing of reinforcing steel should only be permitted when detailed on the plans or authorized by the engineer in writing. Should the contractor desire to hold curb face forms or similar forms with a tie rod welded to the bar steel mat for support, it should only be welded, when permitted, to a bar which is not a part of the structural stress steel, but which is an additional rod tie to the plan reinforcement.

5-15.5.8 Concrete Cover Over Steel

Although coated bar steel is used, the importance of maintaining proper cover for embedment of reinforcing bars cannot be overstressed. If the bars are placed too near the surface of the concrete, particularly in bridge floors, water and deicing chemicals will penetrate the concrete and may cause corrosion of the steel. The steel expands when corroding and soon causes spalling of the surface, which is both unsightly and a costly maintenance problem. Additionally, high steel may cause pop-outs of concrete or result in transverse cracking of decks over reinforcing.

Generally, the plans will show the minimum cover required for reinforcing bars in the bridge floor. If not shown, minimum cover of not less than 2.5" of concrete over the top bars is required. Screed grades should be computed and checked against the height of the bar supports intended to be used, to determine the required cover that needs to be provided. Before placing the concrete, it is absolutely essential project personnel assure themselves the minimum embedment will be obtained. This can be most readily accomplished by attaching a piece of wood, equal in thickness to the minimum required cover, to the screed of the finishing machine and making a preliminary run over the deck. When the rails on which the finishing machine rides are placed on the overhang, the piece of wood should equal the minimum required cover plus the anticipated deflection of the overhang at the rail. The amount of deflection will vary with the width of the overhang, the mass of the load, and the type and spacing of the brackets. However, a general rule of thumb would be to provide for 0.01-foot deflection for each foot of overhang. This preliminary run must be done after the finishing rails have been adjusted for all anticipated deflections. Necessary adjustments should be made in the elevations of the bars.

Inadequate bar steel embedment has occurred on bridge decks most often in the negative moment area over the piers, particularly on prestressed girder structures. Another problem location is at the ends of the heavy longitudinal bars placed over the piers of concrete slab span structures.

The occasional problem with high steel in the vicinity of the piers may have resulted from using the wrong height of supporting chairs. Generally, heavier steel is called for over the pier than in mid-span and, therefore, shorter chairs are required over the piers.

Engineers have experienced tipping or twisting of exterior girders caused by deflection of the overhanging deck when placed. This tipping or twisting is most likely to occur on work with steel or prestressed girders 4 feet or less in depth.

An acceptable practice for stabilizing the exterior girder consists of tying or strapping the upper flanges of the exterior and adjacent girders together and bracing the lower flanges apart. Welding of ties to the upper flanges of steel girders is not an acceptable practice and will not be permitted. Some contractors have welded ties to the shear lugs and this operation must be closely supervised so that no welding occurs on the girder flanges. Even striking an arc on a girder may cause stress concentrations resulting in future fatigue cracking.

Any hardware (except for form ties) not required under the contract but incorporated into portions of the structure where all the reinforcement is corrosion resistant must be galvanized or epoxy-coated steel, stainless steel, or non-metallic materials. Welding is not permitted on epoxy-coated steel.

It is the contractor’s responsibility to determine the need for any bracing or stabilization necessary to prevent girder rotation and overhang settlement.

It is imperative that the potential for girder rotation be carefully evaluated before or at the pre-pour conference in order that any necessary stabilization can be done before deck placement begins.

5-15.6 Placing, Finishing and Curing Concrete (General)

5-15.6.1 Placing Concrete

Before concrete is mixed or ordered for any portion of the structure, the engineer should make sure the contractor has the necessary personnel available, including qualified finishers, and the required items of equipment, such as cranes, buckets, buggies, carts, hoppers, chutes, belts, strike-offs, floats, straightedges, etc., are on hand. The contractor should also ensure that adequate means are available to properly cure the concrete and to protect it from low temperatures. Refer to Figure 1 below for guidelines on what questions to get answered at the prepour meeting.

Figure 1 Concrete Bridge Deck Prepour Meeting Check List

DECK/DECK UNIT of STRUCTURE B-______________________

Project No. __________________

Location of Structure ___________

Date of Conference ___________

Estimated Date of Pour _________

Those Attending Conference:

____

____

____

____

____

____

<u>Pre-Pour Office Questions</u>

- Pour Starting Time __________________________________
- Source of Concrete __________________________________
- No. of Trucks & Rate of Deliver _________________________
- Buckets of Belt Placement _____________________________
- Type of Consolidation (Freq. of Vibrators) __________________
- No. Men on Pour __________________________________
- No. Finishers & Duties _______________________________
- Finishing Tools & Equipment ___________________________
- Application of Surface Texture __________________________
- Timing & Method of Fogging ___________________________
- Emergency Covering Materials ______________________
- Weather Forecast ___________________________________
- Back-up Equipment __________________________________

<u>Pre-Pour Field Operations</u>

Bar Steel Checked for Size ______
Spacing ______
Tieing ______

Dry Run of Finishing Machine Checking Cover ____________________
Fogging Equipment Tried ____________________________________
Check Rails for Pre-Pour Deflection ____________________________
Check Setting of Deck Drains, Expansion Jts., Anchor Bolts ___________

<u>During Pour</u>

- Where can level be set to check rails? __________________________
- Check on Re-Bar Cover & Deck Thickness________________________
- Where can emergency header be placed? ________________________
- Where will Slump & Air Tests be made? _________________________
- Arrangement for Preliminary Air Slump Tests _____________________
- Where can Test Cylinders be placed? ___________________________
- Method of removing Spills, Drips, Mortar, etc., from Girders ____________
- Check setting of Deck Drains, Expansion Jts., Anchor Bolts ___________

<u>Post-Pour</u>

Who will check on Curing, Burlap Cover, etc., ______________________

Placement of high temperature concrete should be avoided. Concrete placed at high temperatures requires higher water content, has a lower ultimate strength and is less durable. To provide for high quality construction during hot weather, [standard spec 501.3.8] should be utilized for bridge, culvert, and retaining wall construction if hot weather concreting (over 80 F concrete temperature) is anticipated.

In essence, the above spec requirements allow the contractor to use concrete with a temperature up to, and including 90 F, if:

- An approved temperature control plan has been submitted, and
- All actions contained in the plan have been followed when concrete temperatures exceed 80 F.

For bridge decks, the contractor also must submit an approved evaporation rate control plan and perform both the actions included in this plan and the actions included in the temperature control plan, while maintaining an evaporation rate of less than or equal to 0.2 lb/sq. ft./hr.

Standard spec 501.3.8 includes the following provisions on paying for ice, based on concrete temperatures:

- If the contractor elects to use ice, as part of their plan to keep concrete temperatures below 85 F, the contractor provides the ice at their expense.
- If concrete temperatures exceed 85 F, the contractor may elect to use ice to control concrete temperatures. The department will pay $0.75 per pound for the ice used in the mix if the contractor has also performed all other actions contained in the temperature control plan.
- If the contractor provides ice solely based on the engineer's orders, the department will pay $0.75 per pound for ice used in the mix, regardless of the concrete temperatures.

While the engineer has the authority to order the contractor to use ice at any time, the engineer should talk to the contractor beforehand about the practicality of obtaining ice, as well as, any other options the contractor may have.

The department's goal is to build new structures with service lives of between 50 and 75 years. To this end, it is very desirable to use concrete with cooler initial mix temperatures. Engineers are encouraged to approve the use of, and payment for, ice according to the spec. When the department pays for ice, according the specs requirements, it is willing to pay for the quantity required to reach a target concrete temperature of 80 F; rather than just pay for the ice needed to keep concrete temperatures below 90 F. The department has determined that the increased quality is worth the relatively small increase in project cost.

There are no negotiable costs. The cost of the contractor's temperature control plan and evaporation rate control plan are incidental to the concrete masonry bid items. The price of ice is fixed at $0.75 per pound.

Some methods for minimizing concrete temperature in hot weather conditions may include:

- Sprinkling aggregate stockpiles with water.
- Shading aggregate stockpiles.
- Insulating or shading water supply lines.
- Using refrigeration chilled water or ice in mix.
- Injecting liquid nitrogen into drum of ready-mix truck after batching.
- If above-ground surge tanks are used for storage of mix water and the stored water has warmed to ambient temperature, drain tanks and refill with cold well water if available.
- Use ready-mix trucks with white or light colored drums to minimize heat gain in transit.
- Sprinkle drums of ready-mix trucks with water after batching and while waiting to discharge at project site.
- Schedule pours in evening, night, or early morning to minimize solar heat gain on aggregate stockpiles, truck drums, and in-place concrete.

Some methods for minimizing the effective evaporation rate during deck placement may include:

- Minimize the concrete temperature by any combination of the above methods.
- Schedule evening, night, or early morning pours when wind is light, air temperature is low and humidity is high.
- Erect windbreaks to minimize effective wind speed.
- Erect fixed fogging systems that will cover the entire pour area to decrease effective air temperature and increase effective humidity (activating individual nozzles after screed passes by).
- Delay pour until more favorable weather conditions resume.

Bridge decks 100 ft and greater in length are cured with a double thickness of wetted burlap. The burlap is to remain in place and be kept thoroughly wet for at least seven days.

There have been several documented cases of spalling at the construction joint between bridge deck and parapet. The spalling appears to have resulted from insufficient vibration and consolidation of the plastic concrete. Consolidation should be complete and uniform to remove trapped air and to achieve full, uniform density. Special attention should also be paid that the vibrator is not used to move the concrete into this small area because segregation will result. The concrete should be placed as closely as possible to its final position before vibration. During a deck pour, it is recommended that vibrators be inserted at a spacing not to exceed a 2-foot by 2-foot. Vibrators never should be dragged laterally through the concrete.

An adequate number of vibrators should be available. Vibrators should be operating at a frequency of at least 7,000 impulses per minute. A quick and accurate field check can be made with a vibrating reed tachometer available through the region office. The tachometer reading shown in revolutions per minute is equivalent to impulses per minute.

The contract change order should include the use of bid item 804.6060 Ice Hot Weather Concreting and $0.75/lb (LB).

5-15.6.2 Removal of Forms and Falsework

Except when forms are permitted to be left in place as provided in standard spec 502.3.4 for concrete placed during cold weather, standard spec 502.3.4.1 requires that forms, other than those forms directly supporting concrete, such as forms under slabs, beams, girders, etc., be removed in from 12 to 72 hours after casting the concrete. Early removal of the forms permits the application of the curing material directly to the concrete, and allows the initial operations of the surface finish of the concrete such as filling holes, pits and other depressions with mortar before the concrete dries out. Better bond, less shrinkage, and greater durability of the patching will be achieved when finish work is performed on young concrete rather than an older concrete.

Early removal of forms during warm weather allows the escape from the concrete of the heat generated by the hydration of the cement. If restricted in its escape, the heat might result in the development of excessive internal temperatures. High internal temperature of the concrete during the hydration period not only produces concrete of lower ultimate strength, but also tends to produce shrinkage cracks.

Furthermore, when fiber pulpboard column forms are used, their removal is much easier at an early date than if left in place for the entire curing period. When forms are removed after the full curing period is completed, the form material tends to separate in layers and stick to the concrete, and the concrete surface does not have as good an appearance as when the forms are removed early.

Fiber pulpboard forms, by nature of their construction, provide an impervious barrier, which to an indefinite degree retains moisture in the concrete, and in some cases, may provide effective curing. Where appearance and durability of the concrete is a prime factor, as is the case with a concrete column, the forms should be removed early and the concrete cured by the membrane method or by a similar method, which provides satisfactory curing and permits proper finishing operations.

Forms that are an integral part of the falsework should not be removed until the falsework can be removed as provided in standard spec 502.3.4.2. During hot weather water should be used, when necessary, to cool the concrete within the forms.

Recently poured concrete masonry when exposed to temperatures at or near freezing experiences little, if any, gain in strength. The spec relating to temperature was written to ensure that forms or falsework would adequately support concrete masonry until it had achieved sufficient strength obtained during favorable curing temperatures.

Removal of forms and falsework from under slabs, beams, girders, etc., which bear the mass of the placed concrete, should be made after the concrete has gained adequate strength to allow their removal. Determination of the time for removal of forms should be made in accordance with the requirements of standard spec 502.3.4.2.

Standard spec 502.3.4.2 provides that for cast in place slab or box girder spans, falsework may not be removed for a minimum period of seven days, regardless of test cylinder strengths and excluding days below 40 F.

5-15.6.3 Finishing Concrete

5-15.6.3.1 Bridge Seats and Anchor Bolts

Special care should be given to the finishing of bridge seats, pier caps, etc., so that full and uniform bearing with the masonry plates will be obtained, and the bearing areas are at the correct elevation. Also, the location of anchor bolts must be accurately laid out before the concrete is cast. If the anchor bolts are to be cast into the pour, they must be rigidly held at the correct position, alignment, and depth of embedment. If the anchor bolts are to be drilled into the hardened concrete and anchored, the position of the bolts should be checked by the use of a template, and the reinforcing steel adjusted as necessary to avoid interference with the bolt locations.

Standard spec 506.3.30 permits anchor bolts to be set in an approved, premixed, non-shrink commercial grout except during freezing weather, or in an epoxy conforming to the standard specs. A listing of approved products is available at:

http://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/tools/appr-prod/default.aspx

5-15.6.3.2 Superstructures

Per standard spec 502.3.6.4, sack-rubbed surface finish must be applied to all exposed formed surfaces of parapets, railings, posts, walks, and curbs and to all exposed side surfaces of superstructures, including the outer face of outside prestressed girders, unless otherwise provided in the special provisions. The prestressed girders must have the finish applied before shipment from the plant.

Rubbed surface finish will not be required for any formed surfaces of superstructures unless required by the plans or special provisions, except that when, in the judgment of the engineer, a satisfactory sack-rubbed finish has not been secured, the contractor must apply a rubbed surface finish, conforming to the requirements of standard spec 502.3.8.3 to the affected exposed areas.

5-15.6.3.3 Substructures

Ordinary surface finish will be required for all formed faces of substructure units, except that parapets and curbs built integrally with the substructure should receive the finish specified for superstructures. Rubbed surface finish will not be required for any formed faces of substructure units unless required by the plans or special provisions or as provided in standard spec 502.3.7.2 for the application of a rubbed surface finish when the ordinary surface finish is defective.

5-15.6.3.4 Sack-rubbed Surface Finish

When so designated on the plans or in the specs, concrete surfaces must have a sack-rubbed finish. Before the application of the sack-rubbed finish, all tie rod holes and large cavities must be filled and all fins and irregularities removed or corrected.

Rubbing the concrete surface with a clean rubber float or fold of burlap and mortar will produce sack-rubbed surface finish. The mortar should consist of one part Portland cement and two parts, by volume, of sand passing a #16 sieve, mixed with sufficient water to provide a consistency of the mortar equivalent to that of a thick cream. The cement used in the mortar should be of the same type and brand used in the concrete. If necessary, to match the surrounding concrete surface, white cement should be blended with the cement.

The surface of the concrete to be finished should be thoroughly wetted and the sack-rubbing performed while the surface is damp but not wet. The mortar should be thoroughly rubbed over the area, filling all pits. While the mortar is still plastic in the pits, the surface should be rubbed using a dry mix of the above proportions, removing all excess plastic material and placing enough dry material in the pits to stiffen and solidify the mortar.

The completed surface should be free of surface voids and blemishes and should be uniform in appearance and texture except for the difference in texture between the filled voids and the remainder of the surface.

5-15.6.3.5 Rubbed Surface Finish

Areas specified to receive a rubbed surface finish require attention to the methods employed and timing of the work. Rubbing must be started as soon as possible after the forms are removed. The surface should be moistened with only enough water so that "lather" can be worked up, and the rubbing should continue until all small holes and depressions are filled. The use of additional mortar should be limited to the filling of cavities produced by the removal of form ties or to pointing-up of honeycombed areas. Plastering of surface with mortar or cement base before or during the rubbing operations should not be permitted. After the rubbing is completed, the surface should not be permitted to dry out until curing is completed, to achieve a durable surface.

5-15.6.4 Curing Concrete

Curing procedures and times for substructures and superstructures are specified in standard spec 502.3.8 and are summarized as follows:

1. For substructures, excluding footings, use applied moisture, wetted burlap, or membrane curing compound.
2. For footings, cure by applied moisture, wetted burlap, membrane curing compound, submersion (if approved), or by leaving the forms in place and then backfilling after removal.
3. For floors, wearing surfaces, and sidewalks, use a fog or fine water spray until hardened, followed by a double layer of wetted burlap. For structures under 100 feet in length polyethylene-coated burlap or other approved coated covering may be used.
4. For inside faces of parapets and railings, use wetted burlap or polyethylene-coated burlap.
5. For outside faces of parapets, railings and exterior girders, use membrane curing compound, wetted burlap or polyethylene-coated burlap.

   One approved curing procedure for substructures, and for the outside faces of parapets, railings, and exterior girders is a clear or translucent membrane curing compound. It is emphasized that this method of curing must be discontinued when a non-uniform or a blotchy and streaked appearance results, and a wetted burlap or moisture cure should be substituted until the cause of the uneven appearance is corrected.

Also, membrane curing compound should not be used before to application of the required surface finish. The other approved curing methods should be used before application of the surface finish.

Membrane curing material should not be applied to construction joints or to surfaces to be bonded to other concrete, nor to surfaces to be waterproofed or sealed.

Membrane curing compounds inhibit the penetration of protective surface sealers and should not be used on bridge decks.

5-15.6.5 Cold Weather Protection

The hardening or setting of concrete is accomplished through a process involving the hydration of the cement. The temperature of the concrete must be within reasonable limits, if normal strength gain is to be expected. Strength gain is very slow below about 40 F.

The standard specs require certain minimum temperatures for fresh concrete placed under cold weather conditions. As the hydration process continues, heat is generated and the temperature of the concrete will increase, normally reaching a maximum within one to three days after placing, then gradually tapering off. The maximum temperature reached and the rate of cooling after passing the peak temperature will depend on the mass of the concrete. In the case of thick, massive sections, or when insulated forms or blankets are protecting the concrete, this build-up of temperature within the concrete may become critical. With elevated temperatures as the concrete is setting, later cooling to normal temperatures may cause shrinkage cracks to occur. Thus, it is important that the internal temperatures of the concrete protected by insulated forms or blankets be closely observed and controlled.

The standard specs provide that structural concrete, placed when the air temperature is 35 F or less, or when an air temperature of 35 F or less can be expected within a period of 6 days following the placing of the concrete, must be protected by housing and heating or by the use of insulated forms or blankets. Unless otherwise provided, this protection is mandatory beginning December 1 and ending March 31. The contractor may delay the erection of housing during the period beginning April 1 and ending November 30 if the air temperature is not expected to fall to 35 F or below during the 24 hours immediately following placement of the concrete. The housing may be delayed until the temperature is expected to fall to 35 F or below during any 24 hours of the succeeding 5-day period, provided that an adequate supply of housing material is maintained at the site and sufficient workers and equipment are available to ensure suitable housing can be erected before the air temperature falls to 35 F.

Standard spec 502.3.7.6 provides that concrete in bridge decks, except railings, parapets and similar pours, cannot be placed when housing is required unless specifically permitted or required by the engineer in writing.

Structural concrete should not be placed without the degree of protection contemplated in the specs unless or until the air temperature has remained above 35 F for a reasonable period and there is at least a short range prediction of air temperatures to remain above 35 F. This, of course, is no assurance air temperatures will not fall below that level within the 6 days following, especially in the early spring and late fall, and the contractor should be required to have materials available to cover or otherwise protect the concrete from damage should this occur.

The engineer may remind the contractor that the contractor is responsible for the proper protection of all concrete exposed to cold weather during the required protection period, and may be required to remove and replace, at the contractor's expense, any concrete damaged by lack of proper protection.

Forms or insulation should be loosened as required and directed by the engineer to the extent that the interior temperature of the concrete not be allowed to exceed 120 F nor fall below 45 F during the protection period, and also that the temperature at the surface of the concrete not be allowed to exceed 100 F. Provision must be made when casting the concrete so internal temperatures can be readily obtained.

Specifications relative to gradual reduction of the temperature of the concrete at the close of the protection period are included in standard spec 501.3.9.

5-15.6.6 Hot Weather Concreting

Special actions are required if concrete temperature at placement is 80 F degrees or more. Standard spec 501.3.8.2 requires the contractor to submit temperature control plans listing potential actions that will be used take to control temperature. Part of the plan may include adding ice to concrete. The engineer should understand the method of bidding of the ice payment item on their given project and administer the required payments accordingly.

The engineer should review the hot weather concreting plan in conjunction to the contractor's construction schedule to assure the need for ice. If the use of ice is the best means to assure quality and critical path, the engineer should not delay concrete pours in order to avoid the payment for ice. The engineer should administer payments to the contractor for ice used if required to meet the requirements listed within the standard specifications. The engineer should not administer payments to the contractor for ice that is delivered and stored on site if it is not required to be used in order to meet the requirements to control concrete temperature as listed within the standard specifications.

5-15.7 Concrete Anchors

Concrete anchors are used to connect concrete elements with other structural or non-structural elements. Anchors can either be cast into concrete (cast in place anchors) or installed after concrete has hardened (post installed anchors). Post installed anchors can be subdivided into either adhesive or mechanical anchors.

Adhesive anchors are used mostly for structure rehabilitation work and pedestrian railing attachments. Rehabilitation work typically uses adhesive anchors to connect existing concrete with new concrete and rebar. The WisDOT LRFD Bridge Manual provides anchor rehabilitation information in section 40.16 of the bridge manual. Railing attachments are usually detailed with cast in place anchors and noted adhesive anchors may be used as an alternative.

Refer to standard spec 502 for other adhesive anchor requirements.

Standard spec 502.2.12 permits approved adhesive anchors when conforming to the standard specs, contract plans, and manufacture requirements. A listing of approved products is available at:

http://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/tools/appr-prod/default.aspx

WisDOT Bureau of Structures (BOS) has placed a moratorium on mechanical anchors due to various concerns.

5-15.7.1 Concrete Anchor Testing Procedure

Tensile pullout loads (proof loads) are used to ensure anchors were installed correctly and satisfy load requirements. Proof loads are not required for cast in place anchors and may or may not be required for adhesive anchors. In most cases, when installed under the direct supervision of an ACI/CRSI certified installer proof loads are not expected. However, proof tests are recommended when field staff have concerns with anchor installation. Proof loads are required when the contractor does not use a certified installer for adhesive anchors. For these cases, anchors are subjected to tensile loads per the standard spec and plan requirements. Refer to standard spec 502 for anchor testing requirements.

Proof loads should be 80 percent of the bar yield load, unless otherwise provided by the contract. Anchor proof loads (ASTM A615) are given in Table 1 for bar steel reinforcement and Table 2 for typical anchor bolts. These values were determined by multiplying 0.80 times the nominal bar area times the bar steel yield strength. In some cases, ASTM minimum proof strengths were used in lieu of bar yield strengths.

Note: Railing attachments are usually located near concrete edges, other anchors, and have shallow embedments. For these cases, it is recommended that the contractor install either cast in place anchors or install adhesive anchors by an ACI/CRSI certified installer. Damage may occur for this application when tested at 80 percent of the bar yield load.

Table 1 Adhesive Anchor Proof Loads for Bar Steel Reinforcement

Bar Size	ASTM A615 (1) GR 60
	Anchor Proof Load (kips)
#3	5.3
#4	9.6
#5	14.9
#6	21.1
#7	28.8
#8	38.0
#9	48.0
#10	61.0

(1) f_y = 60 ksi

Table 2 Adhesive Anchor Proof Loads for Anchor Bolts

Diameter (inches)	ASTM A325 (1)	ASTM F593 (2) (316 Stainless)
	Anchor Proof Load (kips)	
1/2	13.4	10.2
5/8	20.9	16.0
3/4	30.0	15.9
7/8	40.9	21.6
1	53.4	28.3
1 1/8	58.8	35.8
1 1/4	72.6	44.2

(1) fy = Minimum Proof Strength = 85 ksi (1/2" to 1" Dia.)

fy = Minimum Proof Strength = 74 ksi (1 1/8" to 1 1/4" Dia.)

(2) fy = 65 ksi (1/2" to 5/8" Dia.)

fy = 45 ksi (3/4" to 1 1/4" Dia.)