Table of Contents

6.3.4.2 Additional Items to be Destroyed When Construction is Completed (Group B)

1. Miscellaneous correspondence and transmittal letters
2. Preliminary drawings and computations
3. Prints of soil borings and plan profile sheets
4. Shop steel quantity computations*
5. Design checker computations
6. Layout sheets
7. Elevation runs and bridge geometrics
8. Falsework plans*
9. Miscellaneous Test Report
10. Photographs of bridge rehabs

* These items are added to the packet during construction.

6.3.4.3 Items to be Destroyed when Plans are Completed (Group C)

1. All "void" material
2. All copies except one of preliminary drawings
3. Extra copies of plan and profile sheets
4. Preliminary computer design runs

Note that lists for Group A, B & C are not intended to be all inclusive, but serve as starting points for categorizing design material. Items in Group A & B should be labeled separately. Computation of Quantities

6.4 Computations of Quantities

When the final drafting and plan checking is completed, the person responsible for drafting the plans and plans checker are to prepare individual quantity calculations for the bid items listed on the plans. The following instructions apply to the computation on quantities.

Divide the work into units that are repetitive such as footings, columns, and girders. Label all items with a clear description. Use sketches for clarity. These computations may be examined by others in future years so make them understandable.

One of the most common errors made in quantity computation is computing only half of an item which is symmetrical about a centerline and forgetting to double the result.

Following is a list of commonly used bridge quantities. Be sure to use the appropriate item and avoid using incidental items as this is too confusing for the contractor and project manager. The bid item for Abatement of Asbestos Containing Material should be included on the structure plans. Items such as Incentive Strength Concrete Structures, Construction Staking Structure Layout, etc. should not be included on the structure plans.

A column with the title "Bid Item Number" should be the first column for the "Total Estimated Quantities" table shown in the plans. The numbers in this column will be the numbers associated with the bid items as found in the Standard Specification, STSP, and/or Special Provisions.

6.4.1 Excavation for Structures Bridges (Structure)

Bid as Each and as a single unit item for the entire structure. The limits of excavation are shown in the chapter in the manual which pertains to the structural item, abutments, piers, retaining walls, box culverts, etc. If the excavation is required for the roadway, the work may be covered under Excavation Common.

The limits of excavation made into solid rock are the neat line of the footing.

6.4.2 Granular Materials

Granular materials can be bid in units of tons or cubic yards. Structure plans should use the TON bid item for Structure Backfill, Granular Backfill, and Base Aggregate Dense 1 1/4-inch, unless directed otherwise by the Region. The Region may consider use of the CY bid item when contractor-provided tickets may be problematic or when the TON bid item is not used elsewhere on the contract. Other cases may also warrant the use of the CY bid item.

For Structure Backfill, Granular Backfill, and Base Aggregate Dense 1 1/4-inch materials use a 2.0 conversion factor (tons/cubic yard) for compacted TON bid items or use a 1.20 expansion factor (i.e. add an additional 20%) for CY bid items, unless directed otherwise. Refer to the FDM when preparing computations using other granular materials (breaker run, riprap, etc.).

Granular quantities and units should be coordinated with the roadway designer. For some structures, backfill quantities may be negligible to the roadway, while others may encompass a large portion of the roadway cross section and be present in multiple cross sections. A long

6.4.13 Slope Paving Concrete or Slope Paving Crushed Aggregate or Slope Paving Select Crushed Material

Record this quantity to the nearest square yard. Deduct the area occupied by columns or other elements of substructure units.

6.4.14 Riprap Medium, Riprap Heavy or Grouted Riprap, Riprap Light

Record this quantity to the nearest 1 cubic yard.

6.4.15 Pile Points

When recommended in soils report. Bid as each.

6.4.16 Floordrains Type GC, Floordrains Type H, or Floordrains Type WF

Record the type and number of drains. Bid as Each.

6.4.17 Cofferdams (Structure)

Bid as Each and as a single unit item for the entire structure.

6.4.18 Rubberized Membrane Waterproofing

Record the quantity to the nearest square yard.

6.4.19 Expansion Devices

For “Expansion Device” and “Expansion Device Modular”, bid the items in lineal feet. The distance measured is from the outermost extent of the expansion device along the skew (do not include turn-ups into parapets or medians).

6.4.20 Electrical Work

Refer to Standard Construction Specifications for bid items.

6.4.21 Conduit Rigid Metallic __-Inch or Conduit Rigid Nonmetallic Schedule 40 -Inch

Record this quantity in feet.

6.4.22 Preparation Decks Type 1 or Preparation Decks Type 2

Record these quantities to the nearest square yard. Preparation Decks Type 1 should be provided by the Region. Estimate Preparation Decks Type 2 as 40% of Preparation Decks Type 1. Deck preparation areas shall be filled using Concrete Masonry Overlay Decks, Concrete Masonry Deck Repair, or with an appropriate deck patch. See Chapter 40 Standards.

6.4.23 Cleaning Decks

Record this quantity to the nearest square yard.

6.4.24 Joint Repair

Record this quantity to the nearest square yard.

6.4.25 Concrete Surface Repair

Record this quantity to the nearest square foot.

6.4.26 Full-Depth Deck Repair

Record this quantity to the nearest square yard.

6.4.27 Concrete Masonry Overlay Decks

Record this quantity to the nearest cubic yard. Estimate the quantity by using a thickness measured from the existing ground concrete surface to the plan gradeline. Calculate the minimum overlay thickness and add ½” for variations in the deck surface. Provide this average thickness on the plan, as well. Usually 1” of deck surface is removed by grinding. Include deck repair quantities for Preparation Decks Type 1 & 2 and Full-Depth Deck Repair. Use 2-inch thickness for each Preparation area and ½ the deck thickness for Full-Depth Deck Repairs in areas of deck preparation (full-depth minus grinding if no deck preparation).

6.4.28 Removing Structure and Debris Containment

For work over roadways and railroads, “Removing Structure (structure)” is most typically used for complete or substantial removals. For work over waterways, one of the following Standard Specification bid items should be used for complete or substantial removals: Removing Structure Over Waterway Remove Debris (structure); Removing Structure Over Waterway Minimal Debris (structure); or Removing Structure Over Waterway Debris Capture (structure).

For work other than complete or substantial removals, a Removing Structure (structure) bid item may not be required.

Use Debris Containment (structure) bid items, **only as needed** based on the significance, extent, or location of the removal.

See 6.3.3.8 for additional information on Removing Structure and Debris Containment bid items.

Bid as each.

6.4.29 Anchor Assemblies for Steel Plate Beam Guard

Attachment assembly for Beam Guard at the termination of concrete parapets. Bid as each.

Table of Contents

9.1 General

The *Wisconsin Standard Specifications for Highway and Structure Construction* (hereafter referred to as *Standard Specifications*) contains references to *ASTM Specifications* or *AASHTO Material Specifications* which provide required properties and testing standards for materials used in highway structures. The service life of a structure is dependent upon the quality of the materials used in its construction as well as the method of construction. This chapter highlights applications of materials for highway structures and their properties.

In cases where proprietary products are experimentally specified, special provisions are written which provide material properties and installation procedures. Manufacturer's recommendations for materials, preparation and their assistance during installation may also be specified.

Materials that are proposed for incorporation into highway structure projects performed under the jurisdiction of the Wisconsin Department of Transportation (WisDOT) may be approved or accepted by a variety of procedures:

- Laboratory testing of materials that are submitted or samples randomly selected.
- Inspection and/or testing at the source of fabrication or manufacture.
- Inspection and/or testing in the field by WisDOT regional personnel.
- Manufacturer's certificate of compliance and/or manufacturer's certified report of test or analysis, either as sole documentation for acceptance or as supplemental documentation.
- Inspection, evaluation and testing in the normal course of project administration of material specifications.
- Some products are on approved lists or from fabricators, manufacturers, and certified sources approved by WisDOT. Lists of approved suppliers, products, and certified sources are located at:

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

The *Wisconsin Construction and Materials Manual* (CMM) contains a description of procedures for material testing and acceptance requirements in Chapter 8, Section 45. Materials, unless otherwise permitted by the specifications, cannot be incorporated in the project until tested and approved by the proper authority.

9.7 Miscellaneous Materials

Several types of materials are being used as part of a bridge deck protective system. Epoxy coated reinforcing steel, mentioned earlier, is part of this system. Some of these materials or products, are experimental and are placed on specific structures and then monitored and evaluated. A list of materials or products that are part of a bridge deck protective system and are currently used or under evaluation are:

- Galvanized or stainless steel reinforcing bars
- Waterproofing membrane with bituminous concrete overlay
- Microsilica modified concrete or polymer impregnated concrete
- Low slump concrete overlays
- Low-viscosity crack sealer
- Cathodic protection systems with surface overlays

Other materials or products used on highway structures are:

- Downspouts for Type GC and H drains may be fabricated from fiberglass conforming to ASTM D2996, Grade 1, Class A.
- Elastomeric bearing pads (non-laminated) are primarily used with prestressed “I” girders at fixed abutments and piers and at semi-expansion abutments. They are also used with prestressed “slab and box” sections at all supports. The requirements for these pads are described in Section 506.2.6 of the *Standard Specifications*.
- Elastomeric bearing pads (laminated) are primarily used with prestressed “I” girders at expansion supports. The requirements for these pads are described in Section 506.2.6 of the *Standard Specifications*.
- Preformed fillers are placed vertically in the joint between wing and diaphragm in A1 and A5 abutments, in the joint between wing and barrel in box culverts and at expansion joints in concrete cast-in-place retaining walls. Preformed fillers are placed along the front top surface of A1 and A5 abutments, along the outside top surfaces of fixed piers and under flanges between elastomeric bearing pad (non-laminated) and top edge of support. Cork filler is placed vertically on semi-expansion abutments. The requirements for fillers are described in Section 502.2.7 of the *Standard Specifications*.
- Polyethelene sheets are placed on the top surface of semi-expansion abutments to allow movement of the superstructure across the bearing surface. They are also placed between the structural approach slab and the subgrade, and the approach slab and its footing.

- Rubberized waterproofing membranes are used to seal horizontal and vertical joints at the backface of abutments, culverts and concrete cast-in-place retaining walls. See Section 516.2.3 of the *Standard Specifications*.

- Non-staining gray non-bituminous joint sealer is used to seal exposed surfaces of preformed fillers placed in joints as described above. It is also used to place a seal around exposed surfaces of plates used at deflection joints and around railing base plates. The requirement for this joint sealer is referenced in Section 502.2.9 of the *Standard Specifications*.

- Plastic plates may be used at deflection joints in sidewalks and parapets.

- Preformed Fabric, Class A, has been used as a bearing pad under steel bearings. The requirement for this material is given in Section 506.2.8.4 of the *Standard Specifications*.

- Neoprene strip seals are used in single cell and multi-cell (modular) expansion devices.

- Teflon sheets are bonded to steel plates in Type A-T expansion bearings. The requirements for these sheets are found in Section 506.2.8.3 of the *Standard Specifications*.

- Asphalt panels are used on railroad structures to protect the rubber membrane on top of the steel ballast plate from being damaged by the ballast. The requirements for these panels are in the "*Special Provisions*".

- Geotextile fabric is used for drainage filtration, and under riprap and box culverts. This fabric consists of sheets of woven or non-woven synthetic polymers or nylon. Type DF is used for drainage filtration in the pipe underdrain detail placed behind abutments and walls. The fabric allows moisture to drain to the pipe while keeping the backfill from migrating into the coarse material and then into the pipe. Type DF is also used behind abutments or walls that retain soil with backing planks between or behind piling and also for some of the walls detailed in Chapter 14 – Retaining Walls. This fabric will allow moisture to pass through the fabric and the joints in the walls without migration of the soil behind the wall. Type R or HR is placed below riprap and will keep the soil beneath it from being washed away. Type C is placed under breaker run when it is used under box culverts. The requirements for these fabrics are found in Section 645.2 of the *Standard Specifications*.

- Permanent pavement marking types include paint, epoxy, grooved with wet reflective epoxy, and grooved with tape. Grooved pavement markings (0.09 to 0.175 inches deep) are acceptable for concrete bridge decks or concrete overlays but prohibited on thin polymer overlays. Pavement markings on thin polymer overlays should be limited to epoxy markings. Refer to Section 646 of the *Standard Specifications* and Section 650 of the Construction and Materials Manual (CMM) for additional information.

Table of Contents

11.1 General

11.1.1 Overall Design Process

The overall foundation support design process requires an iterative collaboration to provide cost-effective constructible substructures. Input is required from multiple disciplines including, but not limited to, structural, geotechnical and design. For a typical bridge design, the following four steps are required (see 6.2):

1. Structure Survey Report (SSR) – This design step results in a very preliminary evaluation of the structure type and approximate location of substructure units, including a preliminary layout plan.

2. Site Investigation Report – Based on the Structure Survey Report, a Geotechnical Investigation (see Chapter 10 – Geotechnical Investigation) is required, including test borings to determine foundation requirements. A hydraulic analysis is also performed at this time, if required, to assess scour potential and maximum scour depth. The Site Investigation Report and Subsurface Exploration Drawing are used to identify known constraints that would affect the foundations in regard to type, location or size and includes foundation recommendations to support detailed structural design. Certain structure sites/types may require the preliminary structure plans (Step 3) prior to initiating the geotechnical site investigation. One example of this is a multi-span structure over water. See 6.2 for more information.

3. Preliminary Structure Plans – This design step involves preparation of a general plan, elevation, span arrangement, typical section and cost estimate for the new bridge structure. The Site Investigation Report is used to identify possible poor foundation conditions and may require modification of the structure geometry and span arrangement. This step may require additional geotechnical input, especially if substructure locations must be changed.

4. Final Contract Plans for Structures – This design step culminates in final plans, details, special provisions and cost estimates for construction. The Subsurface Exploration sheet(s) are part of the Final Contract Plans. Unless design changes are required at this step, additional geotechnical input is not typically required to prepare foundation details for the Final Contract Plans.

11.1.2 Foundation Type Selection

The following items need to be assessed to select site-specific foundation types:

- Magnitude and direction of loads.
- Depth to suitable bearing material.
- Potential for liquefaction, undermining or scour.
- Frost potential.

- Performance requirements, including deformation (settlement), lateral deflection, global stability and resistance to bearing, uplift, lateral, sliding and overturning forces.

- Ease, time and cost of construction.

- Environmental impact of design and construction.

- Site constraints, including restricted right-of-way, overhead and lateral clearance, construction access, utilities and vibration-sensitive structures.

Based on the items listed above, an assessment is made to determine if shallow or deep foundations are suitable to satisfy site-specific needs. A shallow foundation, as defined in this manual, is one in which the depth to the bottom of the footing is generally less than or equal to twice the smallest dimension of the footing. Shallow foundations generally consist of spread footings but may also include rafts that support multiple columns and typically are the least costly foundation alternative.

Shallow foundations are typically initially considered to determine if this type of foundation is technically and economically viable. Often foundation settlement and lateral loading constraints govern, rather than bearing capacity. Other significant considerations for selection of shallow foundations include requirements for cofferdams, bottom seals, dewatering, temporary excavation support/shoring, over-excavation of unsuitable material, slope stability, available time to dissipate consolidation settlement prior to final construction, scour susceptibility, environmental impacts and water quality impacts. Shallow foundations may not be economically viable when footing excavations exceed 10 to 15 feet below the final ground surface elevation.

When shallow foundations are not satisfactory, deep foundations are considered. Deep foundations can transfer foundation loads through shallow deposits to underlying deposits of more competent deeper bearing material. Deep foundations are generally considered to mitigate concerns about scour, lateral spreading, excessive settlement and satisfy other site constraints.

Common types of deep foundations for bridges include driven piling, drilled shafts, micropiles and augercast piles. Driven piling is the most frequently-used type of deep foundation in Wisconsin. Drilled shafts may be advantageous where a very dense stratum must be penetrated to obtain required bearing, uplift or lateral resistance are concerns, or where obstructions may result in premature driving refusal or where piers need to be founded in areas of shallow bedrock or deep water. A drilled shaft may be more cost effective than driven piling when a drilled shaft is extended into a column and can be used to eliminate the need for a pile footing, pile casing or cofferdams.

Micropiles may be the best foundation alternatives where headroom is restricted or foundation retrofits are required at existing substructures. Micropiles tend to have a higher cost than traditional foundations.

Augercast piles are a potentially cost-effective foundation alternative, especially where lateral loads are minimal. However, restrictions on construction quality control including pile integrity and capacity need to be considered when augercast piles are being investigated. Augercast piles tend to have a higher cost than traditional foundations.

11.1.3 Cofferdams

At stream crossings, tremie-sealed cofferdams are frequently used when footing concrete is required to be placed below the surrounding water level. The tremie-seal typically consists of a plain-cement concrete slab that is placed underwater (in the wet), within a closed-sided cofferdam that is generally constructed of sheetpiling. The seal concrete is placed after the excavation within the cofferdam has been completed to the proper elevation. The seal has three main functions: allowing the removal of water in the cofferdam so the footing concrete can be placed in the dry; serving as a counterweight to offset buoyancy due to differing water elevations within and outside of the cofferdam; and minimizing the possible deterioration of the excavation bottom due to piping and bottom heave. Concrete for tremie-seals is permitted to be placed with a tremie pipe underwater (in-the-wet). Footing concrete is typically required to be placed in-the-dry. In the event that footing concrete must be placed in-the-wet, a special provision for underwater inspection of the footing subgrade is required.

When bedrock is exposed in the bottom of any excavation and prior to placement of tremie concrete, the bedrock surface must be cleaned and inspected to assure removal of loose debris. This will assure good contact between the bedrock and eliminate the potential consolidation of loose material as the footing is loaded.

Cofferdams need to be designed to determine the required sheetpile embedment needed to provide lateral support, control piping and prevent bottom heave. The construction sequence must be considered to provide adequate temporary support, especially when each row of ring struts is installed. Over-excavation may be required to remove unacceptable materials at the base of the footing. Piles may be required within cofferdams to achieve adequate nominal bearing resistance. WisDOT has experienced a limited number of problems achieving adequate penetration of displacement piles within cofferdams when sheetpiling is excessively deep in granular material. Cofferdams are designed by the Contractor.

Refer to 13.11.5 for additional information on cofferdams and seals.

11.1.4 Vibration Concerns

Vibration damage is a concern during construction, especially during pile driving operations. The selection process for the type of pile and hammer must consider the presence of surrounding structures that may be damaged due to high vibration levels. Pile driving operations can cause ground displacement, soil densification and other factors that can damage nearby buildings, structures and/or utilities. Whenever pile-driving operations pose the potential for damage to adjacent facilities (usually when they are located within approximately 100 feet), a vibration-monitoring program should be implemented. This program consists of requiring and reviewing a pile-driving plan submittal, conducting pre-driving and post-driving condition surveys and conducting the actual vibration monitoring with an approved seismograph. A special provision for implementing a vibration monitoring program is available and should be used on projects whenever pile-driving operations or other construction activities pose a potential threat to nearby facilities. Contact the geotechnical engineer for further discussion and assistance, if vibrations appear to be a concern.

11.2 Shallow Foundations

11.2.1 General

Design of a shallow foundation, also known as a spread footing, must provide adequate resistance against geotechnical and structural failure. The design must also limit deformations to within tolerable values. This is true for designs using ASD or LRFD. In many cases, a shallow foundation is the most economical foundation type, provided suitable soil conditions exist within a depth of approximately 0 to 15 feet below the base of the proposed foundation.

WisDOT policy item:

Design shallow foundations in accordance with *AASHTO LRFD*. No additional guidance is available at this time.

Discussion is provided in 12.8 and 13.1 about design loads at abutments and piers, respectively. Live load surcharges at bridge abutments are described in 12.8.

11.2.2 Footing Design Considerations

The following design considerations apply to shallow foundations:

- Scour must not result in the loss of bearing or stability.
- Frost must not cause unacceptable movements.
- External or surcharge loads must be adequately supported.
- Deformation (settlement) and angular distortion must be within tolerable limits.
- Bearing resistance must be sufficient.
- Eccentricity requirements must be satisfied.
- Sliding resistance must be satisfied.
- Overall (global) stability must be satisfied.
- Uplift resistance must be sufficient.
- The effects of ground water must be mitigated and/or considered in the design.

11.2.2.1 Minimum Footing Depth

Foundation type selection and the preliminary design process require input from the geotechnical and hydraulic disciplines. The geotechnical engineer should provide guidance on the minimum embedment for shallow foundations that takes into consideration frost protection and the possible presence of unsuitable foundation materials. The hydraulic engineer should be consulted to assess scour potential and maximum scour depth for water crossings.

At shallow foundations bearing on rock, it is essential to obtain a proper connection to sound rock. Sometimes it is not possible to obtain deep footing embedment in granite or similar hard rock, due to the difficulty of rock removal.

11.2.2.1.1 Scour Vulnerability

Scour is a hydraulic erosion process caused by flowing water that lowers the grade of a water channel or riverbed. For this reason, scour vulnerability is an essential design consideration for shallow foundations. Scour can undermine shallow foundations or remove sufficient overburden to redistribute foundation forces, causing foundation displacement and detrimental stresses to structural elements. Excessive undermining of a shallow foundation leads to gross deformation and can lead to structure collapse.

Scour assessment will require streambed sampling and gradation analysis to define the median diameter of the bed material, D_{50}. Specific techniques for scour assessment, along with a detailed discussion of scour analysis and scour countermeasure design, are presented in the following publications:

- HEC 18 – *Evaluating Scour at Bridges*, 4th Edition
- HEC 20 – *Stream Stability at Highway Structures*, 3rd Edition
- HEC 23 – *Bridge Scour and Stream Instability Countermeasures - Experience, Selection and Design Guidance*, 2nd Edition

Foundations for new bridges and structures located within a stream or river should be located at an elevation below the maximum scour depth that is identified by the hydraulics engineer. In addition, the foundation should be designed deep enough such that scour protection is not required. If the maximum calculated scour depth elevation is below the practical limits for a shallow foundation, a deep foundation system should be used to support the structure.

11.2.2.1.2 Frost Protection

Shallow foundation footings must be embedded below the maximum depth of frost potential (frost depth) whenever frost heave is anticipated to occur in frost-susceptible soil and adequate moisture is available. This embedment is required to prevent foundation heave due to volumetric expansion of the foundation subgrade from freezing and/or to prevent settling due to loss of shear strength from thawing.

Frost susceptible material includes inorganic soil that contains at least 3 percent, dry weight, which is finer in size than 0.02 millimeters. Gravel that contains between 3 and 20 percent fines is least susceptible to frost heave. Bedrock is not considered frost susceptible if the bedrock formation is massive, dense and intact below the footing.

Foundation design is usually not governed by frost heave for foundations bearing on clean gravel and sand or very dense till. Frost heave is a concern whenever the water table, static or perched, is located within 5 feet of the freezing plane.

In Wisconsin, the maximum depth of frost potential generally ranges from approximately 4 feet in the southeastern part of the state to 6 feet in the northwestern corner of the state.

WisDOT policy item:

The minimum depth of embedment of shallow foundations shall be 4 feet, unless founded on competent bedrock.

Further discussion about frost protection in the design of bridge abutments and piers is presented in 12.5 and 13.6, respectively.

11.2.2.1.3 Unsuitable Ground Conditions

Footings should bear below weak, compressible or loose soil. In addition, some soil exhibits the potential for changes in volume due to the introduction or expulsion of water. These volumetric changes can be large enough to exceed the performance limits of a structure, even to the point of structural damage. Both expansive and collapsible soil is regional in occurrence. Neither soil type is well suited for shallow foundation support without a mitigation plan to address the potential of large soil volume changes in this soil, due to changes in moisture content. Expansive and collapsible soils seldom cause problems in Wisconsin.

It should be noted that the procedures presented herein for computing bearing resistance and settlement are applicable to naturally occurring soil and are not necessarily valid for conditions of modified ground such as uncontrolled fills, dumps, mines and waste areas. Due to the unpredictable behavior of shallow foundations in these types of random materials, deep foundations which penetrate through the random material, overexcavation to remove the random material, or subgrade improvement to improve material behavior is required at each substructure unit.

11.2.2.2 Tolerable Movement of Substructures Founded on Shallow Foundations

The bridge designer shall account for any differential settlement (angular distortion) in the design.

WisDOT policy item:

For design of new bridge structures founded on shallow foundations, the maximum permissible movement is 1 inch of horizontal movement at the top of substructure units and 1.5 inches of total estimated settlement of each substructure unit at the Service Limit State.

The sequence of construction can be important when evaluating total settlement and angular distortion. The effects of embankment settlement, as well as settlement due to structure loads, should be considered when the magnitude of total settlement is estimated. It may be possible to manage the settlements after movements have stabilized, by monitoring movements and delaying critical structural connections such as closure pours or casting of decks that are continuous. Generally, project timelines may restrict the time available for soil consolidation. Any project delays for geotechnical reasons must be thoroughly transmitted to, and analyzed by, design personnel.

11.2.2.3 Location of Ground Water Table

The location of the ground water table will impact both the stability and constructability of shallow foundations. A rise in the ground water table will cause a reduction in the effective vertical stress in soil below the footing and a subsequent reduction in the factored bearing resistance. A fluctuation in the ground water table is not usually a bearing concern at depths greater than 1.5 times the footing width below the bottom of footing.

WisDOT policy item:

The highest anticipated groundwater table should be used to determine the factored bearing resistance of footings. The Geotechnical Engineer should select this elevation based on the borings and knowledge of the specific site.

11.2.2.4 Sloping Ground Surface

The influence of a sloping ground surface must be considered for design of shallow foundations. The factored bearing resistance of the footing will be impacted when the horizontal distance is less than three times the footing width between the edge of sloping surface and edge of footing. Shallow foundations constructed in proximity to a sloping ground surface must be checked for overall stability. Procedures for incorporating sloping ground influence can be found in FHWA Publication SH-02-054, *Geotechnical Engineering Circular No. 6 Shallow Foundations* and **LRFD [10.6.3.1.2c]** Considerations for Footings on Slopes.

11.2.3 Settlement Analysis

Settlement should be computed using Service I Limit State loads. Transient loads may be omitted to compute time-dependent consolidation settlement. Two aspects of settlement are important to structural designers: total settlement and differential settlement (ie relative displacement between adjacent substructure units). In addition to the amount of settlement, the designer also needs to determine the time rate for it to occur.

Vertical settlement can be a combination of elastic, primary consolidation and secondary compression movement. In general, the settlement of footings on cohesionless soil, very stiff to hard cohesive soil and rock with tight, unfilled joints will be elastic and will occur as load is applied. For footings on very soft to stiff cohesive soil, the potential for primary consolidation and secondary compression settlement components should be evaluated in addition to elastic settlement.

The design of shallow foundations on cohesionless soil (sand, gravel and non-plastic silt), either as found in-situ or as engineered fill, is often controlled by settlement potential rather than bearing resistance, or strength, considerations. The method used to estimate settlement of footings on cohesionless soil should therefore be reliable so that the predicted settlement is rarely less than the observed settlement, yet still reasonably accurate so that designs are efficient.

Elastic settlement is estimated using elastic theory and a value of elastic modulus based on the results of in-situ or laboratory testing. Elastic deformation occurs quickly and is usually small. Elastic deformation is typically neglected for movement that occurs prior to placement of girders and final bridge connections.

Semi-empirical methods are the predominant techniques used to estimate settlement of shallow foundations on cohesionless soil. These methods have been correlated to large databases of simple and inexpensive tests such as the Standard Penetration Test (SPT) and the Cone Penetrometer Test (CPT).

Consolidation of clays or clayey deposits may result in substantial settlement when the structure is founded on cohesive soil. Settlement may be instantaneous or may take weeks to years to complete. Furthermore, because soil properties may vary beneath the foundation, the duration of the consolidation and the amount of settlement may also vary with the location of the footing, resulting in differential settlement between footing locations. The consolidation characteristics of a given soil are a function of its past history. The reader is directed to FHWA Publication SA-02-054, *Geotechnical Engineering Circular No. 6 Shallow Foundations* for a detailed discussion on consolidation theory and principles.

The rate of consolidation is usually of lesser concern for foundations, because superstructure damage will occur once the differential settlements become excessive. Shallow foundations are designed to withstand the settlement that will ultimately occur during the life of the structure, regardless of the time that it takes for the settlement to occur.

The design of footings bearing on intermediate geomaterials (IGM) or rock is generally controlled by considerations other than settlement. Intermediate geomaterial is defined as a material that is transitional between soil and rock in terms of strength and compressibility, such as residual soil, glacial till, or very weak rock. If a settlement estimate is necessary for shallow foundations supported on IGM or rock, a method based on elastic theory is generally the best approach. As with any of the methods for estimating settlement that use elastic theory, a major limitation is the engineer’s ability to accurately estimate the modulus parameter(s) required by the method.

11.2.4 Overall Stability

Overall stability of shallow foundations that are located on or near slopes is evaluated using a limiting equilibrium slope stability analysis. Both circular arc and sliding-block type failures are considered using a Modified Bishop, simplified Janbu, Spencers or simplified wedge analysis, as applicable. The Service I load combination is used to analyze overall stability. A free body diagram for overall stability is presented in Figure 11.2-1. Maximum resistance factors (or minimum safety factors) for the evaluation of overall stability should be taken as provided in Table 11.2-1.

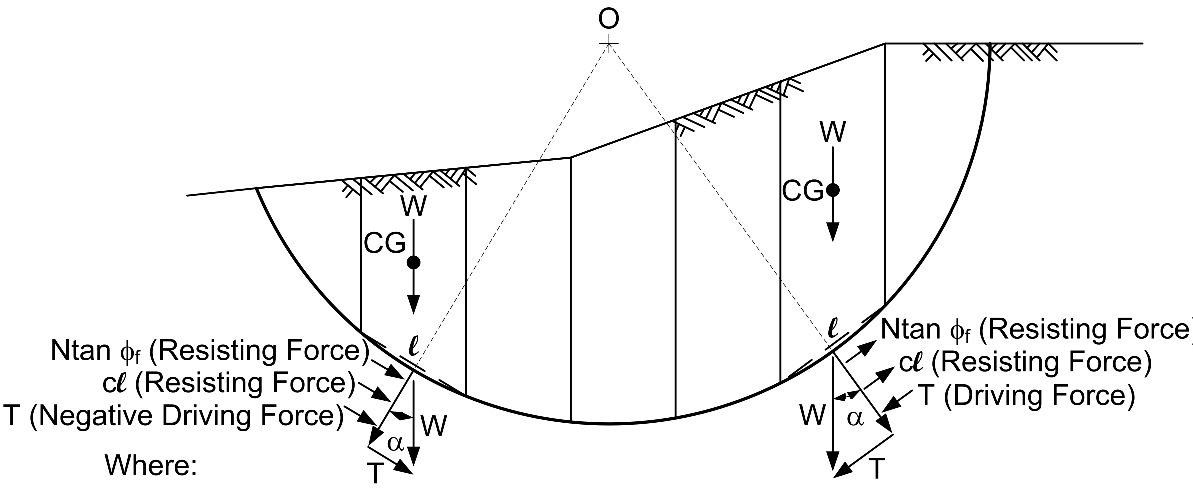
Maximum Resistance Factors (Minimum Safety Factors)		
Slope Configuration	Geotechnical Parameters and Subsurface Stratigraphy Conditions	
	Well Defined	Highly Variable or Based on Limited Information
Slope adjacent to but does not directly support or contain a structural element [2]	**0.75** (1.3)	**0.65** (1.5) [3]
Slope that directly supports or contains a structural element [2]	**0.65** (1.5)	[1]

[1] Contact the Bureau of Technical Services, Geotechnical Engineering Unit
[2] Structural Element: Bridge, retaining wall, critical utility, or other structures with a low tolerance for failure.
[3] For sites with highly variable conditions or limited information that include low strength and/or compressible soils, a lower resistance factor (higher factor of safety) may be applicable. Contact the Bureau of Technical Services, Geotechnical Engineering Unit.

Table 11.2-1
Resistance Factors for Overall Stability Evaluation

Given that resistance factors are used with load factors of 1.0, resistance factors of 0.75 and 0.65 are approximately equivalent to minimum safety factors of 1.3 and 1.5, respectively (i.e. S.F. = 1/ϕ). Until commercially available software is available to analyze overall stability following LRFD methodology, the aforementioned minimum safety factors may be permitted.

Detailed guidance to complete a limiting equilibrium analysis is presented in FHWA Publication NHI-00-045, *Soils and Foundation Workshop Reference Manual* and **LRFD [11.6.2.3]-8th Edition (2017)**. Note: WisDOT has not adopted **LRFD [11.6.3.7]-9th Edition (2020)** with overall stability evaluated at the Strength Limit State.



Where:

CG	=	Center of gravity
c	=	Cohesion along base of slice (psf)
ϕ_f	=	Angle of internal friction of drained soil (degrees)
W	=	Total weight of slice (lb/ft)
N	=	W cos α (lb/ft)
T	=	W sin α (lb/ft)
ℓ	=	length of slice (ft)

Figure 11.2-1
Free Body Diagram for Overall Stability

11.2.5 Footings on Engineered Fills

When shallow foundations are considered for placement on fill, further consideration is required. It is essential to satisfy the design tolerance with regard to total settlement, angular distortion and horizontal movement, including lateral squeeze of the embankment subgrade. The designer must consider the range of probable estimated movement and the impact that this range has on the overall structure performance. The anticipated movement of both new embankment fill and existing embankment materials must be assessed. If shallow foundations are considered, WisDOT requires a thorough subsurface investigation to evaluate settlement of the existing subgrade, including but not limited to continuous soil sampling. WisDOT does not typically place shallow foundations on general embankment fill. WisDOT may consider shallow foundations that are placed on engineered fill, such as that within MSE walls. WisDOT has placed a limited number of shallow foundations on MSE walls for single span bridges. Engineered fill typically consists of high-quality free-draining granular material that is not prone to behavior change due to moisture change, freeze-thaw action, long-term consolidation or shear failure. Engineered fill must also be tightly compacted. On occasion, engineered fill is used in combination with geotextile and/or geogrid to improve shear resistance and overall performance at approach embankments.

If it is not feasible to use a footing to support a sill abutment at the top of slope, it may be feasible to consider a shallow foundation at the bottom of abutment slope to support a full retaining abutment as discussed in 12.2. The increase in stem height will be offset by a reduction in required bridge span length.

11.2.6 Construction Considerations

Shallow foundations require field inspection during construction to confirm that the actual footing subgrade material is equivalent to, or better than, that considered for design. The prepared subgrade should be checked to confirm that the type and condition of the exposed subgrade will provide uniform bearing over the full length or width of footing. The exposed subgrade should be probed to identify possible underlying pockets of soft material that are covered by a thin crust of more competent material. Underlying pockets of soft material and unsuitable material should be over-excavated and replaced with competent material. The structural/geotechnical designer should be contacted if the revised field footing elevations vary by more than one foot lower or three feet higher than the plan elevations, due to differing conditions.

The exposed footing subgrade should be level and stepped, as needed. Stepped shallow foundations may be appropriate when the subsurface conditions vary over the length of substructure unit (footing). For simplicity, planned footing steps should be designated in maximum 4-foot increments. The number and spacing of footing steps is dependent on several factors including, but not limited to, site foundation conditions, temporary excavation support and dewatering requirements, frost and scour depth limitations, constructability, and construction sequence. In general, it is preferred to build uniform step-increments, to simplify construction. Typically the footing with the lowest elevation is constructed first to avoid excavation disturbance of other portions of the footing, as construction continues.

11.2.7 Geosynthetic Reinforced Soil (GRS) Abutment

Geosynthetic Reinforced Soil (GRS) abutments are a type of bridge foundation system typically supporting a single span precast superstructure.. The superstructure is supported on a course-grained soil (gravel) with layers of woven geotextile fabric spaced horizontally from the existing ground, to the base of the slab. The facing is a precast modular block and connected to the woven geotextile fabric. The following reference can be used for design, ‘Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide, Publication Number: FHWA-HRT-11-026’

See 7.1.4.2 for guidance on GRS abutments.

11.3 Deep Foundations

When competent bearing soil is not present near the base of the proposed foundation, structure loads must be transferred to a deeper stratum by using deep foundations such as piles or drilled shafts (caissons). Deep foundations can be composed of piles, drilled shafts, micropiles or augered cast-in-place piles.

The primary functions of a deep foundation are:

- To transmit the load of the structure through a stratum of poor bearing capacity to one of adequate bearing capacity.
- To eliminate objectionable settlement.
- To transfer loads from a structure through erodible soil in a scour zone, to stable underlying strata.
- To anchor structures subjected to hydrostatic uplift or overturning forces.
- To resist lateral loads from earth pressures, as well as external forces.

11.3.1 Driven Piles

Deep foundation support systems have been in existence for many years. The first known pile foundations consisted of rows of timber stakes driven into the ground. Timber piles have been found in good condition after several centuries in a submerged environment. Several types of concrete piles were devised at the turn of the twentieth century. The earliest concrete piles were cast-in-place, followed by reinforced, precast and prestressed concrete piling. The requirement for longer piles with higher bearing capacity led to the use of concrete-filled steel pipe piles in about 1925. More recently, steel H-piles have also been specified due to ease of fabrication, higher bearing capacity, greater durability during driving and the ability to easily increase or decrease driven lengths.

11.3.1.1 Conditions Involving Short Pile Lengths

WisDOT policy generally requires piles to penetrate a minimum of 10 feet through the original ground. Concern exists that short pile penetration in foundation materials of variable consistency may not adequately restrain lateral movements of substructure units. Pile penetrations of less than 10 feet are allowed if prebored at least 3 feet into solid rock. If conditions indicate that minimum pile penetration cannot be achieved, the preboring bid item should be included. Piles which are not prebored into rock must not only meet the 10-foot minimum pile penetration criteria but must also have at least 5 feet of penetration through material with a blow count of at least 7 blows per foot prior to meeting the required driving resistance. Refer to 11.3.1.6 for additional information on preboring.

Refer to 11.3.1.17.6 for additional information on scour considerations.

Foundations without piles (spread footings) should be given consideration at sites where pile penetrations of less than 10 feet are anticipated. The economics of the following two alternatives should be investigated:

1. Design for a shallow foundation founded at a depth where the foundation material is adequate. Embed the footing 6 inches into sound rock for lateral stability.

2. Excavate to an elevation where foundation material is adequate, and backfill to the bottom of footing elevation with suitable granular material or concrete.

11.3.1.2 Pile Spacing

Arbitrary pile spacing rules specifying maximums and minimums are extensively used in foundation design. Proper spacing is dependent upon length, size, shape and surface texture of piles, as well as soil characteristics. A wide spacing of piles reduces heaving and possible uplifting of the pile, damage by tension due to heaving and the possibility of crushing from soil compression. Wider spacing more readily permits the tips of later-driven piles in the group to reach the same depths as the first piles and result in more even bearing and settlement. Large horizontal pressures are created when driving in relatively uncompressible strata, and damage may occur to piles already driven if piles are too closely spaced. In order to account for this, a minimum center-to-center spacing of 2.5 times the pile diameter is often required. **LRFD [10.7.1.2]** calls for a center-to-center pile spacing of not less than 2'-6" or 2.5 pile diameters (widths).

WisDOT policy item:

The minimum pile spacing is 2'-6" or 2.5 pile diameters, whichever is greater. For displacement piles located within cofferdams, or with estimated lengths ≥ 100 ft., the minimum pile spacing is 3.0 pile diameters. The minimum pile spacing for pile-encased piers and pile bents is 3'-0". The maximum pile spacing is 8'-0" for abutments, pile encased piers, and pile bents, based on standard substructure designs.

See Chapter 13 – Piers for criteria on battered piles in cofferdams. The distance from the side of any pile to the nearest edge of footing shall not be less than 9". Piles shall project at least 6" into the footings.

11.3.1.3 Battered Piles

Battered piles are used to resist large lateral loads or when there is insufficient lateral soil resistance within the initial 4 to 5 pile diameters of embedment. Battered piles are frequently used in combination with vertical piles. The lateral resistance of battered piling is a function of the vertical load applied to the pile group. Since the sum of the forces at the pile head must equal zero, increasing the number of battered piles does not necessarily increase the lateral load capacity of the pile group. Both the lateral passive resistance of the soil above the footing as well as the sliding resistance developed at the base of footing are generally neglected in design. See the standard details for further guidance when battered piles are used.

Piles are typically battered at 1 horizontal to 4 vertical. Hammer efficiencies must be reduced when piles are battered. Where negative skin friction loads are anticipated, battered piles should not be considered.

11.3.1.4 Corrosion Loss

Piling should be designed with sufficient corrosion resistance to assure a minimum design life of 75 years. Corrosive sites may included those with combinations of organic soils, high water table, man-made coal combustion products or waste materials, and those materials that allow air infiltration such as wood chips. Experience indicates that corrosion is not a practical problem for steel piles driven in natural soil, due primarily to the absence of oxygen in the soil. However, in fill material at or above the water table, moderate corrosion may occur and protection may be required. Concrete piles are prone to deterioration from exposure to excess concentrations of sulfate and/or chloride. Special consideration (including thicker pile shells, heavier pile sections, painting and concrete encasement) should be given to permanent steel piling that is used in areas of northern Wisconsin which are inhabited by corrosion causing bacteria (see Facilities Development Manual 13-1-15). Typically, WisDOT does not increase pile sections or heavier pile sections to provide corrosion protection outside of these areas.

At potentially corrosive sites, encasement by cast-in-place concrete can provide the required protection for piles extending above the ground surface. All exposed piling should be painted. Additional guidance on corrosion is provided in **LRFD [10.7.5]**.

11.3.1.5 Pile Points

A study was conducted on the value of pile tips (pile points) on steel piles when driving into rock. The results indicated that there was very little penetration difference between the piles driven with pile points and those without. The primary advantages for specifying pile points are for penetrating or displacing boulders, driving through dense granular materials and hardpan layers, and to reduce the potential pile damage in hard driving conditions. Piling can generally be driven faster and in straighter alignment when points are used.

Conical pile points have also been used for round, steel piling (friction and point-bearing) in certain situations. These points can also be flush-welded if deemed necessary.

Standard details for pile points are available from the approved suppliers that are listed in WisDOT's current Product Acceptability List (PAL).

Pile points and preboring are sometimes confused. They are not interchangeable. Pile points can be used to help drive piles through soil that has gravel and/or cobbles or presents other difficult driving conditions. They can also be used to get a good 'bite' when ending piles on sloping bedrock surfaces. Points cannot be used to ensure that piles penetrate into competent bedrock. They may assist in driving through weathered zones of rock or soft rock, but will generally not be effective when penetration into hard rock is desired.

11.3.1.6 Preboring

If embedment into rock is required or minimum pile penetration is doubtful, preboring should be considered. It should be noted that preboring should only be used when appropriate, since many bridge contractors do not own the required construction equipment necessary for this work. Preboring is required for displacement piles when driven into new embankment with fill depths over 10 feet. For problem soils, contact the Bureau of Technical Services, Geotechnical Engineering Unit to discuss preboring considerations.

The following cases may warrant preboring:

- Displacement piles encountering a strong upper stratum with weak underlying soils. If soils (or consistent soil layers) that exhibit SPT refusal (e.g., 50 blows over 6 inches or less) are encountered prior to the scheduled pile tip elevation, pre-boring may be warranted to reduce the risk of unacceptably short pile lengths. Drivability analyses should consider harder than expected intermediate soil layers and be used to determine if preboring is warranted.

- Conditions involving short pile lengths, as discussed in 11.3.1.1. If embedment into rock is required or minimum pile penetration is doubtful, preboring should be considered. For short pile length conditions, piling should be prebored at least 3 feet into solid rock and “firmly seated” on rock after placement in prebored holes. The annular space between the cored rock holes and piling should then be filled with concrete.

Other preboring considerations:

- For displacement piles, preboring should be terminated at least 5 feet above the scheduled pile tip elevation.

- When the pile is planned to be point resistance on rock, preboring may be advanced to plan pile tip elevation. Piles placed in prebored holes founded on rock are typically firmly seated to promote firm contact between pile and rock and do not require driving or restrike to reduce the risk of pile damage.

- The annular space between the prebored hole and piling is required to be backfilled. After the pile is installed, concrete should be used to the top of the rock to properly socket point resistance piles. Clean sand should then be used to backfill the remaining annular space. Backfill material should be deposited with either a tremie pipe or concrete pump to reduce potential arching (bridging) and assure that the complete annular space is filled. Backfill materials for prebored holes should be clearly indicated in the plan documents.

- Some sites may require casing during the preboring operation. If casing is required, it should be clearly indicated in the plan documents.

See 11.3.1.17.6 for scour considerations.

11.3.1.7 Seating

Care must be taken when seating end bearing piles, especially when seating on bedrock with little to no weathered zone. When a pile is firmly seated on rock in prebored holes, pile driving to refusal is not required or recommended, to avoid driving overstress and pile damage. After reaching the predetermined prebore elevation, piles founded in soil are driven with a pile hammer to achieve the specified average penetration or set per blow for the final ten blows of driving.

11.3.1.8 Pile Embedment in Footings

The length of pile embedment in footings is determined based on the type and function of substructure unit and the magnitude of any uplift load.

WisDOT policy item:

Use a minimum 6-inch pile embedment in footings. This embedment depth is considered to result in a free (pinned) head connection for analysis. When the pile embedment depth into the footing is 2.0 feet or greater, the designer can assume a fixed head connection for analysis.

Additional pile embedment is required at some wing walls and at pile-encased substructures, especially where moment connections are required and where cofferdams are not used at stream crossings. Further guidance is provided in 13.6 and in the standard substructure details.

11.3.1.9 Pile-Supported Footing Depth

WisDOT policy item:

Place the bottom of pile-supported footings below the final ground surface at a minimum depth of 2.5 feet for sill abutments, 1.5 feet for sill abutments supported by MSE walls, and 4 feet for piers and other types of abutments.

11.3.1.10 Splices

Full-length piles should be used whenever practical. In no case should timber piles be spliced. Where splices are unavoidable, their number, location and details must be approved by WisDOT prior to pile splicing.

Splice details are shown on Wisconsin bridge plan standards for Pile Details. Splices are designed to develop the full strength of the pile section. Splices should be watertight for CIP concrete piles. Mechanical splice sleeves can be used to join sections of H-pile and pipe pile at greater depth where flexural resistance is not critical. Steel piling 20 feet or less in length is to be furnished in one unwelded piece. Piling from 20 to 50 feet in length can have two shop or field splices, and piling over 50 feet in length can be furnished with up to a maximum of four splices, unless otherwise stated in the project plan documents.

11.3.1.11 Painting

Normally, WisDOT paints all exposed sections of piling. This typically occurs at exposed pier bents.

11.3.1.12 Selection of Pile Types

The selection of a pile type for a given foundation application is made on the basis of soil type, stability under vertical and horizontal loading, long-term settlement, required method of pile installation, substructure type, cost comparison and estimated length of pile. Frequently more than one type of pile meets the physical and technical requirements for a given site. The performance of the entire structure controls the selection of the foundation. Primary considerations in choosing a pile type are the evaluation of the foundation materials and the selection of the substratum that provides the best foundation support.

Piling is generally used at piers where scour is possible, even though the streambed may provide adequate support without piling. In some cases, it is advisable to place footings at greater depths than minimum and specify a minimum pile penetration to guard against excessive scour beneath the footing and piling. Shaft resistance (skin friction) within the maximum depth of scour is assumed to be zero. When a large scour depth is estimated, this area of lost frictional support must be taken into account in the pile driving operations and capacities.

Subsurface conditions at the structure site also affect pile selection and details. The presence of artesian water pressure, soft compressible soil, cobbles and/or boulders, loose/firm uniform sands or deep water all influence the selection of the optimum type of pile for deep foundation support. For instance, WisDOT has experienced ‘running’ of displacement piling in certain areas that are composed of uniform, loose sands. The Department has also experienced difficulty driving displacement piles in denser sands within cofferdams, as consecutive piles are driven, due to compaction of the in-situ sand during pile installation within the cofferdam footprint.

If boulders or cobbles are anticipated within the estimated length of the pile, consideration should be given to increasing the cast-in-place (CIP) pile shell thickness to reduce the potential of pile damage due to high driving stresses. Other alternatives are to investigate the use of pile points or the use of HP piles at the site.

Environmental factors may be significant in the selection of the pile type. Environmental factors include areas subject to high corrosion, bacterial corrosion, abrasion due to moving debris or ice, wave action, deterioration due to cyclic wetting and drying, strong current and gradual erosion of riverbed due to scour. Concrete piles are susceptible to corrosion when exposed to alkaline soil or strong chemicals, especially in rivers and streams. Steel piles can suffer serious electrolysis deterioration if placed in an environment near stray electrical currents. Cast-in-place concrete piling is generally the preferred pile type on structure widenings where displacement piles are required. Timber pile is not to be used, even if timber pile was used on the original structure.

Displacement pile consisting of tapered steel is proprietary and can be an efficient type of friction pile for bearing in loose to medium-dense granular soil. Tapered friction piles may need to be installed with the aid of water jetting in dense granular soil. Straight-sided friction piles are recommended for placement in cohesive soils underlain by a granular stratum to develop the greatest combined shaft and point resistance. Steel HP or open-end pipe piles are best suited for driving through obstructions or fairly competent layers to bedrock. Foundations such as pier bents which may be subject to large lateral forces when located in deep and/or swiftly moving water require piles that can sustain large bending forces. Precast, prestressed concrete pile is best suited for high lateral loading conditions but is seldom used on Wisconsin transportation projects.

11.3.1.12.1 Timber Piles

Current design practice is not to use timber piles.

11.3.1.12.2 Concrete Piles

The three principal types of concrete pile are cast-in-place (CIP), precast reinforced and prestressed reinforced. CIP concrete pile types include piles cast in driven steel shells that remain in-place, and piles cast in unlined drilled holes or shafts. Driven-type concrete pile is discussed below in this section. Concrete pile cast in drilled holes is discussed later in this chapter and include drilled shafts (11.3.2), micropiles (11.3.3), and augered cast-in-place piles (11.3.4).

Depending on the type of concrete pile selected and the foundation conditions, the load-carrying capacity of the pile can be developed by shaft resistance, point resistance or a combination of both. Generally, driven concrete pile is employed as a displacement type pile.

When embedded in the earth, plain or reinforced concrete pile is generally not vulnerable to deterioration. The water table does not affect pile durability provided the concentration level is not excessive for acidity, alkalinity or chemical salt. Concrete pile that extends above the water surface is subject to abrasion damage from floating objects, ice debris and suspended solids. Deterioration can also result from frost action, particularly in the splash zone and from concrete spalling due to internal corrosion of the reinforcement steel. Generally, concrete spalls are a concern for reinforced concrete pile more than prestressed pile because of micro-cracks due to shrinkage, handling, placement and loading. Prestressing reduces crack width. Concrete durability increases with a corresponding reduction in aggregate porosity and water/cement ratio. WisDOT does not currently use prestressed reinforced concrete pile.

11.3.1.12.2.1 Driven Cast-In-Place Concrete Piles

Driven cast-in-place (CIP) concrete piles are formed by pouring concrete into a thin-walled closed-end steel shell which has been previously driven into the ground. A flat, oversize plate is typically welded to the bottom of the steel shell. Steel shells are driven either with or without a mandrel, depending on the wall thickness of the steel shell and the shell strength that is required to resist driving stress. Piling in Wisconsin is typically driven without the use of a mandrel. The minimum thickness of the steel shell should be that required for pile reinforcement and to resist driving stress. The Contractor may elect to furnish steel shells with greater thickness to permit their choice of driving equipment. A thin-walled shell must be carefully evaluated so that it does not collapse from soil pressure or deform from adjacent pile driving. Deformities or distortions in the pile shell could constrict the flow of concrete into the pile and produce voids or necking that reduce pile capacity. It is standard construction practice to inspect the open shell prior to concrete placement. Care must be exercised to avoid intermittent voids over the pile length during concrete placement.

Driven CIP concrete piles are considered a displacement-type pile, because the majority of the applied load is usually supported by shaft resistance. This pile type is frequently employed in slow flowing streams and areas requiring pile lengths of 50 to 120 feet. Driven CIP pile is generally selected over timber pile because of the availability of different diameters and wall thicknesses, the ability to adjust driven lengths and the ability to achieve greater resistances.

Driven CIP concrete piles may have a uniform cross section or may be tapered. The minimum cross-sectional area is required to be 100 and 50 square inches at the pile butt and tip, respectively. The Department has only used a limited number of tapered CIP piles and has experienced some driving problems with them.

For consistency with WisDOT design practice, the steel shell is ignored when computing the axial structural resistance of driven CIP concrete pile that is symmetrical about both principal axes. This nominal (ultimate) axial structural resistance capacity is computed using the following equation, neglecting the contribution of the steel shell to resist compression: **LRFD [Eq'n 5.6.4.4-3]**.

$$P_u \leq P_r = \phi P_n$$

Where:

$$P_n = 0.80(k_C \cdot f'_c \cdot (A_g - A_{st})) + f_y \cdot A_{st}$$

Where:

P_u = Factored axial force effect (kips)

P_r = Factored axial resistance without flexure (kips)

ϕ = Resistance factor

P_n = Nominal axial resistance without flexure (kips)

A_g = Gross area of concrete pile section (inches^2)

A_{st} = Total area of longitudinal reinforcement (inches^2)

k_C = Ratio of max. concrete compressive stress to specified compressive strength of concrete; $k_C = 0.85$ (for $f'_c \leq 10.0$ ksi)

f_y = Specified yield strength of reinforcement (ksi)

f'_c = Concrete compressive strength (ksi)

For cast-in-place concrete piles with steel shell and no steel reinforcement bars, A_{st} equals zero and the above equation reduces to the following.

$$P_n = 0.68f'_c A_g$$

A resistance factor, ϕ, of 0.75 is used to compute the factored structural axial resistance capacity, as specified in **LRFD [5.5.4.2]**. For CIP piling there are no reinforcing ties, however the steel shell acts to confine concrete similar to ties.

$$P_r = 0.51f_c A_g$$

For piles subject to large lateral loads, the structural pile capacity must also be checked for shear and combined stress against flexure and compression.

Piles subject to uplift must also be checked for tension resistance.

A concrete compressive strength of 4 ksi is the minimum value required by specification, while a value of 3.5 ksi is used in the structural design computations. Pile capacities are maximums, based on an assumed concrete compressive strength of 3.5 ksi. The concrete compressive strength of 3.5 ksi is based on construction difficulties and unknowns of placement. The Geotechnical Site Investigation Report must be used as a guide in determining the nominal geotechnical resistance for the pile.

Any structural strength contribution associated with the steel shell is neglected in driven CIP concrete pile design. Therefore, environmentally corrosive sites do not affect driven CIP concrete pile designs. An exception is that CIP should not be used for exposed pile bents in corrosive environments as shown in the *Facilities Development Manual,* Procedure 13-1-15.

Based on the above equation, current WisDOT practice is to design driven cast-in-place concrete piles for factored (ultimate structural) axial compression resistances as shown in Table 11.3-5. See 6.3.2.1 for the typical style of plan notes showing axial resistance as well as required driving resistance on plans. **If less than the maximum axial resistance is required by design, state only the required corresponding driving resistance on the plans.** The minimum shell thickness is 0.219 inches for straight steel tube and 0.1793 inches for fluted steel shells, unless otherwise noted in the Geotechnical Site Investigation Report and stated in the project plans. Exposed piling (e.g. open pile bents) should not be less than 12 inches in diameter.

When cobbles or other difficult driving conditions are present, the minimum wall thickness for steel shells of driven cast-in-place concrete piles should be increased to 0.25 inches or thicker to facilitate driving without damaging the pile. A drivability analysis should be completed in design, to determine the required wall thickness based on site conditions and an assumed driving equipment.

Driven cast-in-place concrete pile is generally the most favorable displacement pile type since inspection of the steel shell is possible prior to concrete placement and more reliable control of concrete placement is attainable.

11.3.1.12.2.2 Precast Concrete Piles

Precast concrete pile can be divided into two primary types – reinforced concrete piles and prestressed concrete piles. These piles have parallel or tapered sides and are usually of rectangular or round cross section. Since the piles are usually cast in a horizontal position, the round cross section is not common because of the difficulty involved in filling a horizontal cylindrical form. Because of the somewhat variable subsurface conditions in Wisconsin and the need for variable length piles, these piles are currently not used in Wisconsin.

11.3.1.12.3 Steel Piles

Steel pile generally consist of either H-pile or pipe pile types. Both open-end and closed-end pipe pile are used. Pipe piles may be left open or filled with concrete, and can also have a structural shape or reinforcement steel inserted into the concrete. Open-end pipe pile can be socketed into bedrock with preboring.

Steel pile is typically top driven at the pile butt. However, closed-end pipe pile can also be bottom driven with a mandrel. Mandrels are generally not used in Wisconsin.

Steel pile can be used in friction, point-bearing, a combination of both, or rock-socketed piles. One advantage of steel pile is the ease of splicing or cutting to accommodate differing final constructed lengths.

Steel pile should not be used for exposed pile bents in corrosive environments as show in the *Facilities Development Manual, Procedure 13.1.15*.

The nominal (ultimate) axial structural compressive resistance of steel piles is designed in accordance with **LRFD [10.7.3.13.1]** as either non-composite or composite sections. Composite sections include concrete-filled pipe pile and steel pile that is encased in concrete. The nominal structural compressive resistance for non-composite and composite steel pile is further specified in **LRFD [6.9.4 and 6.9.5]**, respectively. The effective length of horizontally unsupported steel pile is determined in accordance with **LRFD [10.7.3.13.4]**. Resistance factors for the structural compression limit state are specified in **LRFD [6.5.4.2]**.

WisDOT policy item:

For steel H-piles, 50 ksi yield strength material shall be used. For steel pipe piles, 45 ksi yield strength material shall be used. Plans shall note specified yield strength.

11.3.1.12.3.1 H-Piles

Steel piles are generally used for point-bearing piles and typically employ what is known as the HP-section (often called H-piles for brevity). Steel H-piles are rolled sections with wide flanges such that the depth of the section and the width of the flanges are approximately equal.

The cross-sectional area and volume displacement are relatively small and as a result, H-piles can be driven through compact granular materials and slightly into soft rock. Also, steel piles have little or no effect in causing ground swelling or raising of adjacent piles. Because of the small volume of H-piles, they are considered “non-displacement” piling.

H-piles are available in many sizes and lengths. Unspliced pile lengths up to 140 feet and spliced pile lengths up to 230 feet have been driven. Typical pile lengths range from 40 to 120 feet. Common H-pile sizes vary between 10 and 14 inches.

The current WisDOT practice is to design driven H-piles for the factored (ultimate structural) axial compression resistance as shown in Table 11.3-5. These values are based on ϕ_c = 0.5 for severe driving conditions **LRFD [6.5.4.2]**. See 6.3.2.1 for the typical style of plan notes showing axial resistance as well as required driving resistance on plans. **If less than the maximum axial resistance is required by design, state only the required corresponding driving resistance on the plans.**

Since granular soil is largely incompressible, the principal action at the tip of the pile is lateral displacement of soil particles. Although it is an accepted fact that steel piles develop extremely high loads per pile when driven to point-bearing on rock, some misconceptions still remain that H-piles cannot function as friction piles. Load tests indicate that steel H-piles can function quite satisfactorily as friction piles in sand, sand-clay, silt-and-sand or hard clay. However, they are not as efficient as displacement piles in these conditions and typically drive to greater depths. The surface area for pile frictional computations is considered to be the projected “box area” of the H-pile, and not the actual steel surface area.

Clay is compressible to a far greater degree than sand or gravel. As the solid particles are pressed into closer contact with each other and water is squeezed out of the voids, only small frictional resistance to driving is generated because of the lubricating action of the free water. However, after driving is completed, the lateral pressure against the pile increases due to dissipation of the pore water pressures. This causes the fine clay particles to increase adherence to the comparatively rough surface of the pile. Load is transferred from the pile to the soil by the resulting strong adhesive bond. In many types of clay, this bond is stronger than the shearing resistance of the soil.

In hard, stiff clays containing a low percentage of voids and pore water, the compressibility is small. As a result, the amount of displacement and compression required to develop the pile’s full capacity are correspondingly small. As an H-pile is driven into stiff clay, the soil trapped between the flanges and web usually becomes very hard due to the compression and is carried down with it. This trapped soil acts as a plug and the pile can also act as a displacement pile.

In cases where loose soil is encountered, considerably longer point-bearing steel piles are required to carry the same load as relatively short displacement-type piles. This is because a displacement-type pile, with its larger cross section, produces more compaction as it is driven through materials such as soft clays or loose organic silts. H-piles are not typically used in exposed pile bents due to concerns with debris catchment.

11.3.1.12.3.2 Pipe Piles

Pipe piles consist of seamless, welded or spiral welded steel pipes in diameters ranging from 7-3/4 to 24 inches. Other sizes are available, but they are not commonly used. Typical wall thicknesses range from 0.375-inch to 0.75-inch, with wall thicknesses of up to 1.5 inches possible. Pipe piles should be specified by grade with reference to ASTM A 252.

Pipe piles may be driven either open or closed end. If the end bearing capacity from the full pile toe area is required, the pile toe should be closed with a flat plate or a conical tip.

11.3.1.12.3.3 Oil Field Piles

The oil industry uses a very high quality pipe in their drilling operations. Every piece is tested for conformance to their standards. Oil field pipe is accepted as a point-bearing alternative to HP piling, provided the material in the pipe meets the requirements of ASTM A 252, Grade 3, with a minimum tensile strength of 120 ksi or a Brinell Hardness Number (BHN) of 240, a minimum outside diameter of 7-3/4 inches and a minimum wall thickness of 0.375-inch. The weight and area of the pipe shall be approximately the same as the HP piling it replaces. Sufficient bending strength shall be provided if the oil field pipe is replacing HP piling in a pile bent. Oil field pipe is driven open-ended and not filled with concrete. The availability of this pile type varies and is subject to changes in the oil industry.

11.3.1.12.4 Pile Bents

See 13.1 for criteria to use pile bents at stream crossings. When pile bents fail to meet these criteria, pile-encased pier bents should be considered. To improve debris flow, round piles are generally selected for exposed bents. Round or H-piles can be used for encased bents.

11.3.1.13 Tolerable Movement of Substructures Founded on Driven Piles

WisDOT policy item:

For design of new bridge structures founded on driven piles, limit the horizontal movement at the top of the foundation unit to 0.5 inch or less at the service limit state.

11.3.1.14 Resistance Factors

The nominal (ultimate) geotechnical resistance capacity of the pile should be based on the type, depth and condition of subsurface material and ground water conditions reported in the Geotechnical Site Investigation Report, as well as the method of analysis used to determine pile resistance. Resistance factors to compute the factored geotechnical resistance are presented in **LRFD [Table 10.5.5.2.3-1]** and are selected based on the method used to determine the nominal (ultimate) axial compression resistance. The design intent is to adjust the resistance factor based on the reliability of the method used to determine the nominal pile resistance. When construction controls, are used to improve the reliability of capacity prediction (such as pile driving analyzer or static load tests), the resistance factors used during final design should be increased in accordance with **LRFD [Table 10.5.5.2.3-1]** to reflect planned construction monitoring.

WisDOT exception to AASHTO:

WisDOT requires at least four (4) piles per group to support each substructure unit, including each column for multi-column bents. WisDOT does not reduce geotechnical resistance factors to satisfy redundancy requirements to determine axial pile resistance. Hence, redundancy resistance factors in **LRFD [10.5.5.2.3]** are not applicable to WisDOT structures. This exception applies to typical CIP concrete pile and H-pile foundations. Non-typical foundations (such as drilled shafts) shall be investigated individually.

No guidance regarding the structural design of non-redundant driven pile groups is currently included in *AASHTO LRFD*. Since WisDOT requires a minimum of 4 piles per substructure unit, structural design should be based on a load modifier, η, of 1.0. Further description of load modifiers is presented in **LRFD [1.3.4]**.

The following geotechnical resistance factors apply to the majority of the Wisconsin bridges that are founded on driven pile. On the majority of WisDOT projects, wave equation analysis and dynamic monitoring are not used to set driving criteria. This equates to typical resistance factors of 0.35 to 0.45 for pile design. A summary of resistance factors is presented in Table 11.3-1, based on **LRFD [Table 11.5.5.2.3-1]**, which are generally used for geotechnical design on WisDOT projects.

<table>
<tr><th colspan="3">Condition/Resistance Determination Method</th><th>Resistance Factor</th></tr>
<tr><td rowspan="7">Static Analysis – Used in Design Phase</td><td rowspan="3">Nominal Resistance of Single Pile in Axial Compression, φ_{stat}</td><td>Skin Friction and End Bearing in Clay and Mixed Soil
Alpha Method</td><td>0.35</td></tr>
<tr><td>Skin Friction and End Bearing in Sand
Nordlund/Thurman Method</td><td>0.45</td></tr>
<tr><td>Point Bearing in Rock</td><td>0.45</td></tr>
<tr><td>Block Failure, φ_{bl}</td><td>Clay</td><td>0.60</td></tr>
<tr><td rowspan="2">Uplift Resistance of Single Pile, φ_{up}</td><td>Clay and Mixed Soil
Alpha Method</td><td>0.25</td></tr>
<tr><td>Sand
Nordlund Method</td><td>0.35</td></tr>
<tr><td>Horizontal Resistance of Single Pile or Pile Group</td><td>All Soil Types and Rock</td><td>1.0</td></tr>
<tr><td colspan="2" rowspan="2">Nominal Resistance of Single Pile in Axial Compression – Dynamic Analysis –</td><td>FHWA-modified Gates dynamic pile formula (end of drive condition only)</td><td>0.50 (1)</td></tr>
<tr><td>Wave equation analysis, without pile dynamic measurements or load test, at end of drive condition only</td><td>0.50</td></tr>
</table>

for the Hammer and Pile Driving System Actually - used During Construction for Pile Installation, φ_{dyn}	Driving criteria established by dynamic test [Pile Driving Analyzer, (PDA)] with signal matching [CAse Pile Wave Analysis Program, (CAPWAP)] at beginning of redrive conditions only, of at least two production pile per substructure, but no less than 2% of the structure production piles. Quality control of remaining piles by calibrated wave equation and/or dynamic testing.	0.65
	Static Pile Load Test(s) and dynamic test (PDA) with signal matching (CAPWAP) at beginning of redrive conditions only, of at least two production pile per substructure, but no less than 2% of the structure production piles. Quality control of remaining piles by calibrated wave equation and/or dynamic testing.	0.80

(1) Based on department research and past experience

Table 11.3-1
Geotechnical Resistance Factors for Driven Pile

Resistance factors for structural design of piles are based on the material used, and are presented in the following sections of *AASHTO LRFD*:

- Concrete piles – **LRFD [5.5.4.2]**
- Steel piles – **LRFD [6.5.4.2]**

11.3.1.15 Bearing Resistance

A pile foundation transfers load into the underlying strata by either shaft resistance, point resistance or a combination of both. Any driven pile will develop some amount of both shaft and point resistance. However, a pile that receives the majority of its support capacity by friction or adhesion from the soil along its shaft is referred to as a friction pile, whereas a pile that receives the majority of its support from the resistance of the soil near its tip is a point resistance (end bearing) pile.

The design pile capacity is the maximum load the pile can support without exceeding the allowable movement criteria. When considering design capacity, one of two items may govern the design – the nominal (ultimate) geotechnical resistance capacity or the structural resistance capacity of the pile section. This section focuses primarily on the geotechnical resistance capacity of a pile.

The factored load that is applied to a single pile is carried jointly by the soil beneath the tip of the pile and by the soil around the shaft. The total factored load is not permitted to exceed the factored resistance of the pile foundation for each limit state in accordance with **LRFD [1.3.2.1 and 10.7.3.8.6]**. The factored bearing resistance, or pile capacity, of a pile is computed as follows:

$$\sum \eta_i \gamma_i Q_i \leq R_r = \varphi R_n = \varphi_{stat} R_p + \varphi_{stat} R_s$$

Where:

η_i	=	Load modifier
γ_i	=	Load factor
Q_i	=	Force effect (tons)
R_r	=	Factored bearing resistance of pile (tons)
R_n	=	Nominal resistance (tons)
R_p	=	Nominal point resistance of pile (tons)
R_s	=	Nominal shaft resistance of pile (tons)
φ	=	Resistance factor
φ_{stat}	=	Resistance factor for driven pile, static analysis method

This equation is illustrated in Figure 11.3-1.

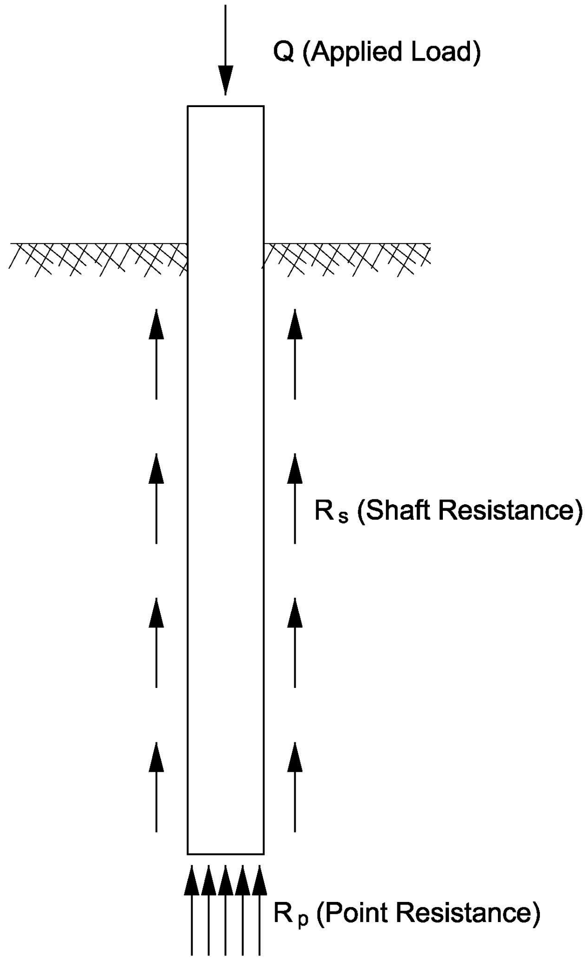


Figure 11.3-1
Resistance Distribution for Axially-Loaded Pile

11.3.1.15.1 Shaft Resistance

The shaft resistance of a pile is estimated by summing the frictional resistance developed in each of the different soil strata.

For non-cohesive (granular) soil, the total shaft resistance can be calculated using the following equation (based on the Nordlund/Thurman Method):

$$R_s = \Sigma C_d D K_\delta C_F \sigma_v' \frac{\sin(\delta + \omega)}{\cos(\omega)}$$

Where:

R_s = Total shaft resistance capacity (tons)

C_d = Pile perimeter (feet)

D = Pile segment length (feet)

K_δ = Coefficient of lateral earth pressure at mid-point of soil layer under consideration from **LRFD [Figures 10.7.3.8.6f-1 through 10.7.3.8.6f-4]**

C_F = Correction factor for K_δ when $\delta \neq \phi_f$, from **LRFD [Figure 10.7.3.8.6f-5],** whereby ϕ_f = angle of internal friction for drained soil

σ_v' = Effective overburden pressure at midpoint of soil layer under consideration (tsf)

δ = Friction angle between the pile and soil obtained from **LRFD [Figure 10.7.3.8.6f-6]** (degrees)

ω = Angle of pile taper from vertical (degrees)

For cohesive (fine-grained) soil, the total shaft resistance can be calculated using the following equation (based on the alpha method):

$$R_s = \Sigma\, \alpha\, S_u\, C_d\, D$$

Where:

R_s = Total (nominal) shaft resistance capacity (tons)

α = Adhesion factor based on the undrained shear strength from **LRFD [Figure 10.7.3.8.6b-1]**

S_u = Undrained shear strength (tsf)

C_d = Pile perimeter (feet)

D = Pile segment length (feet)

Typical values of nominal shaft resistance for various soils are presented in Table 11.3-2 and Table 11.3-3. The values presented are average ranges and are intended to provide orders of magnitude only. Other conditions such as layering sequences, drilling information, ground water, thixotropy and clay sensitivity must be evaluated by experienced geotechnical engineers and analyzed using principles of soil mechanics.

Soil Type	q_u (1) (tsf)	Nominal Shaft Resistance (psf)
Very soft clay	0 to 0.25	---
Soft clay	0.25 to 0.5	200 to 450
Medium clay	0.5 to 1.0	450 to 800
Stiff clay	1.0 to 2.0	800 to 1,500
Very stiff clay	2.0 to 4.0	1,500 to 2,500
Hard clay	4.0	2,500 to 3,500
Silt	---	100 to 400
Silty clay	---	400 to 700
Sandy clay	---	400 to 700
Sandy silt	---	600 to 1,000
Dense silty clay	---	900 to 1,500

(1) Unconfined Compression Strength

Table 11.3-2
Typical Nominal Shaft Resistance Values for Cohesive Material

Soil Type	N_{160} (1)	Nominal Shaft Resistance (psf)
Very loose sand and silt or clay	0 to 6	50 to 150
Medium sand and silt or clay	6 to 30	400 to 600
Dense sand and silt or clay	30 to 50	600 to 800
Very dense sand and silt or clay	over 50	800 to 1,000
Very loose sand	0 to 4	700 to 1,700
Loose sand	4 to 10	700 to 1,700
Firm sand	10 to 30	700 to 1,700
Dense sand	30 to 50	700 to 1,700
Very dense sand	over 50	700 to 1,700
Sand and gravel	---	1,000 to 3,000
Gravel	---	1,500 to 3,500

(1) Standard Penetration Value (AASHTO T206) corrected for both overburden and hammer efficiency effects (blows per foot).

Table 11.3-3
Typical Nominal Shaft Resistance Values for Granular Material

Shaft resistance values are dependent upon soil texture, overburden pressure and soil cohesion but tend to increase with depth. However, experience in Wisconsin has shown that shaft resistance values in non-cohesive materials reach constant final values at depths of 15 to 25 pile diameters in loose sands and 25 to 35 pile diameters in firm sands.

In computing shaft resistance, the method of installation must be considered as well as the soil type. The method of installation significantly affects the degree of soil disturbance, the lateral stress acting on the pile, the friction angle and the area of contact. Shafts of prebored piles do not always fully contact the soil; therefore, the effective contact area is less than the shaft surface area. Driving a pile in granular material densifies the soil and increases the friction angle. Driving also displaces the soil laterally and increases the horizontal stress acting on the pile. Disturbance of clay soil from driving can break down soil structure and increase pore pressures, which greatly decreases soil strength. However, some or all of the strength recovers following reconsolidation of the soil due to a decrease in excess pore pressure over time. Use the initial soil strength values for design purposes. The type and shape of a pile also affects the amount of shaft resistance developed, as described in 11.3.1.12.

11.3.1.15.2 Point Resistance

The point resistance, or end bearing capacity, of a pile is estimated from modifications to the bearing capacity formulas developed for shallow footings.

For non-cohesive soils, point resistance can be calculated using the following equation (based on the Nordlund/Thurman Method):

$$R_p = A_p \, \alpha_t \, N'_q \, \sigma_v' \leq q_L \, A_p$$

Where:

R_p = Point resistance capacity (tons)

A_p = Pile end area (feet^2)

α_t = Dimensionless factor dependent on depth-width relationship from **LRFD [Figure 10.7.3.8.6f-7]**

N'_q = Bearing capacity factor from **LRFD [Figure 10.7.3.8.6f-8]**

σ_v' = Effective overburden pressure at the pile point $\leq$ 1.6 (tsf)

q_L = Limiting unit point resistance from **LRFD [Figure 10.7.3.8.6f-9]** (tsf)

For cohesive soils, point resistance can be calculated using the following equation:

$$R_p = 9 S_u A_p$$

Where:

R_p = Point resistance capacity (tons)

S_u = Undrained shear strength of the cohesive soil near the pile base (tsf)

A_p = Pile end area (feet^2)

This equation represents the maximum value of point resistance for cohesive soil. This value is often assumed to be zero because substantial movement of the pile tip (1/10 of the pile diameter) is needed to mobilize point resistance capacity. This amount of tip movement seldom occurs after installation.

A point resistance (or end bearing) pile surrounded by soil is not a structural member like a column. Both experience and theory demonstrate that there is no danger of a point resistance pile buckling due to inadequate lateral support if it is surrounded by even the very softest soil. Therefore, pile stresses can exceed column stresses. Although, exposed pile bent piles may act as structural columns.

11.3.1.15.3 Group Capacity

The nominal resistance capacity of pile groups may be less than the sum of the individual nominal resistances of each pile in the group for friction piles founded in cohesive soil. For pile groups founded in cohesive soil, the pile group must be analyzed as an equivalent pier for block failure in accordance with **LRFD [10.7.3.9]**. WisDOT no longer accepts the Converse-Labarre method of analysis to account for group action. If the pile group is tipped in a firm stratum overlying a weak layer, the weak layer should be checked for possible punching failure in accordance with **LRFD [10.6.3.1.2a]**. Experience in Wisconsin indicates that in most thixotropic clays where piles are driven to a hammer bearing as determined by dynamic formulas, pile group action is not the controlling factor to determine pile resistance capacity. For pile groups in sand, the sum of the nominal resistance of the individual piles always controls the group resistance.

11.3.1.16 Lateral Load Resistance

Structures supported by single piles or pile groups are frequently subjected to lateral forces from lateral earth pressure, live load forces, wave action, ice loads and wind forces. Piles subjected to lateral forces must be designed to meet combined stress and deflection criteria to prevent impairment or premature failure of the foundation or superstructure. To solve the soil-structure interaction problems, the designer must consider the following:

- Pile group configuration.
- Pile stiffness.
- Degree of fixity at the pile connection with the pile footing.

- Maximum bending moment induced on the pile from the superstructure load and moment distribution along the pile length.
- Probable points of fixity near the pile tip.
- Soil response (P-y method) for both the strength and service limit states.
- Pile deflection permitted by the superstructure at the service limit state.

If a more detailed lateral load investigation is desired, a P-y analysis is typically performed using commercially available software such as COM624P, FB Multi-Pier or L-Pile. A resistance factor of 1.0 is applied to the soil response when performing a P-y analysis using factored loads since the soil response represents a nominal (ultimate) condition. For a more detailed analysis of lateral loads and displacements, refer to the listed FHWA design references at the end of this chapter or a geotechnical engineering book.

<u>WisDOT policy item:</u>

A detailed analysis is required for the lateral resistance of piles used in A3 abutments.

11.3.1.17 Other Design Considerations

Several other topics should be considered during design, as presented below.

11.3.1.17.1 Downdrag Load

This section is currently under development to address recent changes within the AASHTO LRFD Bridge Design Specifications (Tenth Edition - 2024). Until further guidance is provided, the Neutral Plane Method may be used in lieu of the aforementioned section. Refer to FHWA GEC 12 (2016) Section 7.3.6.1 for information on the neutral plane method design approach. Contact the Bureau of Technical Services, Geotechnical Engineering Unit for additional assistance.

Negative shaft resistance (downdrag) results in the soil adhesion forces pulling down the pile instead of the soil adhesion forces resisting the applied load. This can occur when settlement of the soil through which the piling is driven takes place. It has been found that only a small amount of settlement is necessary to mobilize these additional pile (drag) loads. This settlement occurs due to consolidation of softer soil strata caused by such items as increased embankment loads (due to earth fill) or a lowering of the existing ground water elevation. The nominal pile resistance available to support structure loads plus downdrag shall be estimated by considering only the positive skin and tip resistance below the lowest layer acting to produce negative skin resistance. When this condition is present, the designer may provide time to allow consolidation to occur before driving piling, or **LRFD [10.7.3.8.6]** may be used to estimate the available pile resistance to withstand the downdrag plus structure loads. Other alternatives are to pre-auger the piling, drive the pile to bearing within a permanent pipe sleeve that is placed from the base of the substructure unit to the bottom of the soft soil layer(s), coat the pile with bitumen above the compressible soil strata or use proprietary materials to encase the piles (within fill constructed after the piling is installed). The Department has experienced problems with bitumen coatings.

The factored axial compression resistance values given for H-piles in Table 11.3-5 are conservative and based on Departmental experience to avoid overstressing during driving. For H-piles in end bearing, loading from downdrag is allowed in addition to the normal pile loading, since this is a post-driving load. Use the values given in Table 11.3-5 and design piling as usual. Additionally, up to 45, 60, and 105 tons downdrag for HP 10x42, HP 12x53, and HP 14x73 piles respectively is allowed when the required driving resistance is determined by the modified Gates formula.

11.3.1.17.2 Lateral Squeeze

Lateral squeeze as described in **LRFD [10.7.2.6]** occurs when pile supported abutments are constructed on embankments and/or MSE walls over soft soils. Typically, the piles are installed prior to completion of the embankment and/or MSE wall, and therefore are potentially subject to subsurface soil instability. If the embankment and/or MSE wall has a marginal factor of safety with regards to slope stability, then lateral squeeze has the potential to laterally deflect the piles and tilt the abutment. Typically, if the shear strength of the subsurface soil is less than the height of the embankment times the unit weight of the embankment divided by three, then damage from lateral squeeze could be expected.

If this is a potential problem, the following are the recommended solutions from the *FHWA Design and Construction of Driven Piles Manual*:

1. Delay installation of abutment piling until after settlement has stabilized (best solution).

2. Provide expansion shoes large enough to accommodate the movement.

3. Use steel H-piles strong enough and rigid enough to provide both adequate strength and deflection control.

4. Use lightweight fill to reduce driving forces.

11.3.1.17.3 Uplift Resistance

Uplift forces may also be present, both permanently and intermittently, on a pile system. Such forces may occur from hydrostatic uplift or cofferdam seals, ice uplift resulting from ice grip on piles and rising water, wind uplift due to pressures against high structures or frost uplift. In the absence of pulling test data, the calculated factored shaft resistance should be used to determine static uplift capacity to demand ratio (CDR). A minimum CDR value of 1.0 is required. Generally, the type of pile with the largest perimeter is the most efficient in resisting uplift forces.

11.3.1.17.4 Pile Setup and Relaxation

The nominal resistance of a deep foundation may change over time, particularly for driven piles. The nominal resistance may increase (setup) during dissipation of excess pore pressure, which developed during pile driving, as soil particles reconsolidate after the soil has been remolded during driving. The shaft resistance may decrease (relaxation) during dissipation of negative pore pressure, which was induced by physical displacement of soil during driving. If the potential for soil relaxation is significant, a non-displacement pile is preferred over a displacement type pile. Relaxation may also occur as a result of a deterioration of the bearing stratum following driving-induced fracturing, especially for point-bearing piles founded on non-durable bedrock. Relaxation is generally associated with densely compacted granular material.

Pile setup has been found to occur in some fine-grained soil in Wisconsin. Pile setup should not be included in pile design unless pre-construction load tests are conducted to determine site-specific setup parameters. The benefits of obtaining site-specific setup parameters could include shortening friction piles and reducing the overall foundation cost. Pile driving resistance would need to be determined at the end of driving and again later after pore pressure dissipation. Restrike tests involve additional taps on a pile after the pile has been driven and a waiting period (generally 24 to 72 hours) has elapsed. The dynamic monitoring analysis are used to predict resistance capacity and distribution over the pile length.

CAPWAP(CAse Pile Wave Analysis Program) is a signal matching software. CAPWAP uses dynamic pile force and velocity data to discern static and dynamic soil resistance, and then estimate static shaft and point resistance for driven pile. Pile top force and velocity are calculated based on strain and acceleration measurements during pile driving, with a pile driving analyzer (PDA). CAPWAP is based on the wave equation model which characterizes the pile as a series of elastic beam elements, and the surrounding soil as plastic elements with damping (dynamic resistance) and stiffness (static resistance) properties.

Typically, a test boring is drilled and a static load test is performed at test piles where pile setup properties are to be determined. Typical special provisions have been developed for use on projects incorporating aspects of pile setup. Pile setup is discussed in greater detail in FHWA Publication NHI-05-042, *Design and Construction of Driven Pile Foundations*.

Restrike tests with an impact hammer can be used to identify change in pile resistance due to pile setup or relaxation. Restrike is typically performed by measuring pile penetration during the first 10 blows by a warm hammer. Due to setup, it is possible that the hammer used for initial driving may not be adequate to induce pile penetration and a larger hammer may be required to impart sufficient energy for restrike tests. Only warm hammers should be used for restrikes by first applying at least 20 blows to another pile.

Restrike tests with an impact hammer must be used to substantiate the resistance capacity and integrity of pile that is initially driven with a vibratory hammer. Vibratory hammers may be used with approval of the engineer. Other than restrikes with an impact hammer, no formula exists to reliably predict the resistance capacity of a friction pile that is driven with a vibratory hammer.

11.3.1.17.5 Drivability Analysis

In order for a driven pile to develop its design geotechnical resistance, it must be driven into the ground without damage. Stresses developed during driving often exceed those developed under even the most extreme loading conditions. The critical driving stress may be either compression, as in the case of a steel H-pile, or tension, as in the case of a concrete pile.

Drivability is treated as a strength limit state. The geotechnical engineer will perform the evaluation of this limit state during design based on a preliminary dynamic analysis using wave equation techniques. These techniques are used to document that the assumed pile driving hammers are capable of mobilizing the required nominal (ultimate) resistance of the pile at driving stress levels less than the factored driving resistance of the pile. Drivability can often be the controlling strength limit state check for a pile foundation. This is especially true for high capacity piles driven to refusal on rock.

Drivability analysis is required by **LRFD [10.7.8]**. A drivability evaluation is needed because the highest pile stresses are usually developed during driving to facilitate penetration of the pile to the required resistance. However, the high strain rate and temporary nature of the loading during pile driving allow a substantially higher stress level to be used during installation than for service. The drivability of candidate pile-hammer-system combinations can be evaluated using wave equation analyses.

As stated in the 2004 FHWA Design and Construction of Driven Pile Foundations Manual:

> "The wave equation does not determine the capacity of the pile based on soil boring data. The wave equation calculates a penetration resistance for an assumed ultimate capacity, or conversely it assigns estimated ultimate capacity to a pile based upon a field observed penetration resistance."

> "The accuracy of the wave equation analysis will be poor when either soil model or soil parameters inaccurately reflect the actual soil behavior, and when the driving system parameters do not represent the state of maintenance of hammer or cushions."

The following presents potential sources of wave equation errors.

- Hammer Data Input, Diesel Hammers
- Cushion Input
- Soil Parameter Selection

LRFD [C10.7.8] states that the local pile driving results from previous drivability analyses and historical pile driving experience can be used to refine current drivability analyses. WisDOT recommends using previous pile driving records and experience when performing and evaluating drivability analyses. These correlations with past pile driving experience allow modifications of the input values used in the drivability analysis, so that results agree with past construction findings.

Driving stress criteria are specified in the individual LRFD material design sections and include limitations of unfactored driving stresses in piles based on the following:

- Yield strength in steel piles, as specified in **LRFD [6.4.1]**
- Ultimate compressive strength of the gross concrete section, accounting for the effective prestress after losses for prestressed concrete piles loaded in tension or compression, as specified in **LRFD [5.6.4.4]**

Though there are a number of ways to assess the drivability of a pile, the steps necessary to perform a drivability analysis are typically as follows:

1. Estimate the total resistance of all soil layers. This may include layers that are not counted on to support the completed pile due to scour or potential downdrag, but will have to be driven through. WisDOT recommends using the values for quake and damping provided in the FHWA Design and Construction of Driven Pile Foundations Manual.

   In addition, the soil resistance parameters should be reduced by an appropriate value to account for the loss of soil strength during driving. The following table provides some guidelines based on Table 9-19 of the FHWA Design and Construction of Driven Pile Foundations Manual:

Soil Type	Recommended Soil Set Up Factor[1]	Percentage Loss of Soil Strength during Driving
Clay	2.0	50 percent
Silt – Clay	1.5[2]	33 percent
Silt	1.5	33 percent
Sand – Clay	1.5	33 percent
Sand – Silt	1.2	17 percent
Fine Sand	1.2	17 percent
Sand	1.0	0 percent
Sand - Gravel	1.0	0 percent

Notes:
1. Confirmation with local experience recommended
2. The value of 1.5 is higher than the FHWA Table 9-19 value of 1.0 based upon WisDOT experience.

Table 11.3-4
Soil Resistance Factors

   Incorporation of loss of soil strength and soil set-up should only be accounted for in the pile drivability analyses. Typically, WisDOT does not include set-up in static pile design analyses.

2. Select a readily available hammer. The following hammers have been used by Wisconsin Bridge Contractors: Delmag D-12-42, Delmag D-12-32, Delmag D-12, Delmag D-15, Delmag D-16-32, Delmag D-19, Delmag D-19-32, Delmag D-19-42, Delmag D-25, Delmag D-30-32, Delmag D-30, Delmag D-36, MKT-7, Kobe K-13, Gravity Hammer 5K.

3. Model the driving system, soil and pile using a wave equation program. The driving system generally includes the pile-driving hammer, and elements that are placed between the hammer and the top of pile, which include the helmet, hammer cushion,

and pile cushion (concrete piles only). Pile splices are also modeled. Compute the driving stress using the drivability option for the wave equation, which shows the pile compressive stress an blow counts versus depth for the given soil profile.

4. Determine the permissible driving stress in the pile. During the design stage, it is often desirable to select a lower driving stress than the maximum permitted. This will allow the contractors greater flexibility in hammer selection. WisDOT generally limits driving stress to 90 percent of the steel yield strength

5. Evaluate the results of the drivability analysis to determine a reasonable blow count (that is, ranges from 25 blows per foot to 120 blows per foot) associated with the permissible driving stress.

The goal of the drivability study is to evaluate the potential for excessive driving stresses and to determine that the pile/soil system during driving will result in reasonable blow counts. The drivability study is not intended to evaluate the ultimate pile capacity or establish plan lengths. If the wave equation is used to set driving criteria, then contact the Bureau of Technical Services, Geotechnical Engineering Unit to discuss the proper procedures.

11.3.1.17.6 Scour

If a substructure unit is located in a stream or lake, consideration should be given to the effects of the anticipated stream bed scour when selecting the footing type. During design, estimated pile lengths may require an increase to compensate for scour loss. The scour depth is estimated and used to compute the estimated shaft resistance that is lost over the scour depth (exposed pile length). The required pile length is then increased to compensate for the resistance capacity that is lost due to scour. The pile length is increased based on the following equation:

$$R_n = R_{n\text{-stat}} + R_{n\text{-scour}}$$

Where:

R_n	=	Nominal shaft resistance capacity, adjusted for scour effect (tons)
$R_{n\text{-stat}}$	=	Nominal shaft resistance based on static analysis, without scour consideration (tons)
$R_{n\text{-scour}}$	=	Nominal shaft resistance lost (negative value) over the exposed pile length due to scour (tons)

The Site Investigation Report shall determine if preboring is necessary. Additionally, Special Provisions and/or plan notes may also be necessary to address unique preboring requirements. This may include, but is not limited to indicating minimum pile embedments, minimum pile tip elevations, and clarifying payment for preboring.

WisDOT policy item:

If there is potential for scour at a site, account for the loss of pile resistance from the material within the scour depth. The designer must not include any resistance provided by this material when determining the nominal pile resistance. Since the material within the scour depth may be present during pile driving operations, the additional resistance provided by this material shall be considered when determining the required driving resistance. The designer should also consider minimum pile tip elevation requirements.

11.3.1.17.7 Typical Pile Resistance Values

Table 11.3-5 shows the typical pile resistance values for several pile types utilized by the Department. The table shows the Nominal Axial Compression Resistance (Pn), which is a function of the pile materials, the Factored Axial Compression Resistance (Pr), which is a function of the construction procedures, and the Required Driving Resistance, which is a function of the method used to measure pile capacity during installation. The bridge designer uses the Factored Axial Compression Resistance to determine the number and spacing of the piles. The Required Driving Resistance is placed on the plans. See 6.3.2.1-7 for details regarding plan notes.

Pile Size	Shell Thickness (inches)	Concrete or Steel Area (A_g or A_s) (in^2)	Nominal Resistance (Pn) (tons) (2)(3)(6)	(ϕ)	Maximum Factored Resistance (Pr) (tons) (4)	Modified Gates Driving Criteria		PDA/CAPWAP Driving Criteria	
						Factored Resistance (Pr) (ϕ = 0.50) (tons)	Required Driving Resistance (Rn_{dyn}) (tons) (5)	Factored Resistance (Pr) (ϕ = 0.65) (tons)	Required Driving Resistance (Rn_{dyn}) (tons) (5)
Cast in Place Piles									
10 ¾“	0.219	83.5	99.4	0.75	75	55 (8)	110 (11)	72 (8)	110 (11)
10 ¾“	0.250	82.5	98.2	0.75	74	65 (8)	130 (11)	75 (9)	115
10 ¾“	0.365	78.9	93.8	0.75	70	75 (9)	150	75 (9)	115
10 ¾“	0.500	74.7	88.8	0.75	67	75 (9)	150	75 (9)	115
12 ¾”	0.250	118.0	140.4	0.75	105	80 (8)	160 (11)	104 (8)	160 (11)
12 ¾”	0.375	113.1	134.6	0.75	101	105 (9)	210	104 (9)	160
12 ¾”	0.500	108.4	129.0	0.75	97	105 (9)	210	104 (9)	160
14”	0.250	143.1	170.3	0.75	128	85 (8)	170 (11)	111 (8)	170 (11)
14”	0.375	137.9	164.1	0.75	123	120 (8)	240 (11)	120	185
14”	0.500	132.7	158.0	0.75	118	120 (9)	240	120 (9)	185
16”	0.375	182.6	217.3	0.75	163	145 (8)	290 (11)	159	245
16”	0.500	176.7	210.3	0.75	158	160 (9)	320	159 (9)	245
H-Piles									

10 x 42	NA[1]	12.4	310.0	0.50	155	90	180 [10]	117	180 [10]
12 x 53	NA[1]	15.5	387.5	0.50	194	110	220 [10]	143	220 [10]
14 x 73	NA[1]	21.4	535.0	0.50	268	125	250 [10]	162	250 [10]

Table 11.3-5

Typical Pile Axial Compression Resistance Values

Notes:

1. NA – not applicable

2. For CIP Piles: Pn = 0.8 (k_C * f'c * Ag + fy * As) **LRFD [Eq'n 5.6.4.4-3]**. k_C = 0.85 (for $f'_C \leq$ 10.0 ksi). Neglecting the steel shell, equation reduces to 0.68 * f'c * Ag.

   f'c = compressive strength of concrete = 3,500 psi

3. For H-Piles: Pn = (0.66^{λ} * Fe * As) **LRFD [Eq'n 6.9.5.1-1]** (λ = 0 for piles embedded in the ground below the substructure, i.e. no unsupported lengths)

   Fe = fy = yield strength of steel = 50,000 psi

4. Pr = ϕ * Pn

   ϕ = 0.75 (**LRFD [5.5.4.2]** for axial compression concrete)

   ϕ = 0.50 (**LRFD [6.5.4.2]** for axial steel, for difficult driving conditions)

5. The Required Driving Resistance is the lesser of the following:

   - $Rn_{dyn} = Pr / \varphi_{dyn}$

     φ_{dyn} = 0.50 for construction driving criteria using modified Gates

     φ_{dyn} = 0.65 for construction driving criteria using PDA/CAPWAP

   - The nominal required driving resistance is based on past experience. For H-Piles, refer to note 10. For CIP Piles, refer to note 11.

6. Values for Axial Compression Resistance are calculated assuming the pile is fully supported. Piling not in the ground acts as an unbraced column. Calculations verify that the pile values given in Table 11.3-5 are valid for open pile bents within the limitations described in 13.2.2. Cases of excessive scour require the piling to be analyzed as unbraced columns above the point of streambed fixity.

7. If less than the maximum axial resistance, P_r, is required by design, state only the required corresponding driving resistance on the plans.

8. The Factored Axial Compression Resistance is controlled by the maximum allowable driving resistance based on 70 percent of the specified yield strength of steel rather than concrete capacity. Refer to note 11 for additional information.

9. Values were rounded up to the value above so as to not penalize the capacity of the thicker walled pile of the same diameter. (Wisconsin is conservative in not considering the pile shell in the calculation of the Factored Axial Compression Resistance. Rounded values utilize some pile shell capacity)

10. Rn_{dyn} values given for H-Piles are representative of past Departmental experience (rather than Pn x ø) and are used to avoid problems associated with overstressing during driving. These Rn_{dyn} values utilize 46 to 58 percent of the specified yield strength, which is less than the drivability limit **[LRFD 10.7.8]**. If other H-Piles are utilized that are not shown in the table, values should be held to approximately this same range.

11. Rn_{dyn} values given for CIP piles are representative of past Departmental experience of using 35 ksi yield strength material and are used to avoid problems associated with overstressing during driving. These Rn_{dyn} values utilize 70 percent (90% x 35ksi/45ksi) of specified yield strength, which is less than the drivability limit **[LRFD 10.7.8]**. If other CIP Piles are utilized that are not shown in the table, values should be held to the same limit.

11.3.1.18 Construction Considerations

Construction considerations generally include selection of pile hammers, use of driving formulas and installation of test piles, when appropriate, as described below.

11.3.1.18.1 Pile Hammers

Pile driving hammers are generally powered by compressed air, steam pressure or diesel units. The diesel hammer, a self-contained unit, is the most popular due to its compactness and adoption in most construction codes. Also, the need for auxiliary power is eliminated and the operation cost is nominal. Vibratory and sonic type hammers are employed in special cases where speed of installation is important and/or noise from impact is prohibited. The vibrating hammers convert instantly from a pile driver to a pile extractor by merely tensioning the lift line.

Pile hammers are raised and allowed to fall either by gravity or with the assistance of power. If the fall is due to gravity alone, the hammer is referred to as single-acting. The single-acting hammer is suitable for all types of soil but is most effective in penetrating heavy clays. The major disadvantage is the slow rate of driving due to the relatively slow rate of blows from 50 to 70 per minute. Wisconsin construction specifications call for a minimum hammer weight depending on the required final bearing value of the pile being driven. In order to avoid damage to the pile, the fall of the gravity hammer is limited to 10 feet.

If power is added to the downward falling hammer, the hammer is referred to as double-acting. This type of hammer works best in sandy soil but also performs well in clay. Double-acting hammers deliver 100 to 250 blows per minute, which increases the rate of driving considerably over the single-acting hammers. Wisconsin construction specifications call for a rated minimum energy of 15 percent of the required bearing of the pile. A rapid succession of blows at a high velocity can be extremely inefficient, as the hammer bounces on heavy piles.

Differential-acting hammers overcome the deficiencies found with both single- and double-acting hammers by incorporating higher frequency of blows and more efficient transfer of energy. The steam cycle, which is different from that of any other hammer, makes the lifting area under the piston independent of the downward thrusting area above the piston. Sufficient force can be applied for lifting and accelerating these parts without affecting the dead weight needed to resist the reaction of the downward acceleration force. The maximum delivered energy per blow is the total weight of the hammer plus the weight of the downward steam force times the length of the stroke.

The contractor’s selection of the pile hammer is generally dependent on the following:

- The hammer weight and rated energy are selected on the basis of supplying the maximum driving force without damaging the piles.

- The hammer types dictated by the construction specification for the given pile type.

- The hammer types available to the contractor.

- Special situations, such as sites adjacent to existing buildings, that require consideration of vibrations generated from the driving impact or noise levels. In these instances, reducing the hammer size or choosing a double-acting hammer may be preferred over a single-acting hammer. Impact hammers typically cause less ground vibration than vibratory hammers.

- The subsurface conditions at the site.

- The required final resistance capacity of the pile.

WisDOT specifications require the heads of all piling to be protected by caps during driving. The pile cap serves to protect the pile, as well as modulate the blows from the hammer which helps eliminate large inefficient hammer forces. When penetration-per-blow is used as the driving criteria, constant cap-block material characteristics are required. The cap-block characteristics are also assumed to be constant for all empirical formula computations to determine the rate of penetration equivalent to a particular dynamic resistance.

11.3.1.18.2 Driving Formulas

Formulas used to estimate the bearing capacity of piles are of four general types – empirical, static, dynamic and wave equation.

Empirical formulas are based upon tests under limited conditions and are not suggested for general use.

Static formulas are based on soil stresses and try to equate shaft resistance and point resistance to the load-bearing capacity of the piles.

Dynamic pile driving formulas assume that the kinetic energy imparted by the pile hammer is equal to the nominal pile resistance plus the energy lost during driving, starting with the following relationship:

$$\text{Energy input} = \text{Energy used} + \text{Energy lost}$$

The energy used equals the driving resistance multiplied by the pile movement. Thus, by knowing the energy input and estimating energy losses, driving resistance can be calculated from observed pile movement. Numerous dynamic formulas have been proposed. They range from the simpler Engineering News Record (ENR) Formula to the more complex Hiley Formula. A modified Engineering News Formula was previously used by WisDOT to determine pile resistance capacity during installation. All new designs shall use the FHWA-modified Gates dynamic pile formula (modified Gates) or WAVE equation for determining the required driving resistance.

The following modified Gates formula is used by WisDOT:

$$R_R = \varphi_{dyn} R_{ndr} = \varphi_{dyn}\left(0.875\left(E_d\right)^{0.5}\log_{10}\left(10/s\right) - 50\right)$$

Where:

R_R	=	Factored pile resistance (tons)
φ_{dyn}	=	Resistance factor = 0.50, as specified in Table 11.3-1
R_{ndr}	=	Nominal pile resistance measured during pile driving (tons)
E_d	=	Energy delivered by the hammer per blow (lb-foot)
s	=	Average penetration in inches per blow for the final 10 blows (inches/blow)

Because of the difficulty of evaluating the many energy losses involved with pile driving, these dynamic formulas can only approximate pile driving resistance. These approximate results can be used as a safe means of determining pile length and bearing requirements. Despite the obvious limitations, the dynamic pile formulas take into account the best information available and have considerable utility to the engineer in securing reasonably safe and uniform results over the entire project.

The wave equation can be used to set driving criteria to achieve a specified pile bearing capacity (contact the Bureau of Technical Services, Geotechnical Engineering Unit prior to using the wave equation to set the driving criteria). The wave equation is based upon the theory of longitudinal wave transmission. This theory, proposed by Saint Venant a century ago, did not receive widespread use until the advent of computers due to its complexity. The wave equation can predict impact stresses in a pile during driving and estimate static soil resistance at the time of driving by solving a series of simultaneous equations. An advantage of this method is that it can accommodate any pile shape, as well as any distribution of pile shaft resistance and point resistance. The effect of the hammer and cushion block can be included in the computations.

Dynamic monitoring is performed by a Pile Driving Analyzer (PDA). WisDOT uses the PDA to evaluate the driving criteria, which is set by a wave equation analysis, and in an advisory capacity for evaluating if sufficient pile penetration is achieved, if pile damage has occurred or if the driving system is performing satisfactorily.

The PDA provides a method of dynamic pile testing both for pile design and construction control. Testing is accomplished during pile installation by attaching reusable strain transducers and accelerometers directly on the pile. Piles can be tested while being driven or during restrike. The instrumentation mounted on the pile allows the measurement of force and acceleration signals for each hammer blow. This data is transmitted to a small field computer for processing and recording. Calculations made by the computer based upon one-dimensional wave mechanics provide an immediate readout of maximum stresses in the pile, energy transmitted to the pile and a prediction of the nominal axial resistance of the pile for each hammer impact. Monitoring of the force and velocity wave traces with the computer during driving also enables detection of any structural pile damage that may have occurred. Review of selected force and velocity wave traces are also available to provide additional testing documentation. The PDA can be used on all types of driven piles with any impact type of pile-driving hammer.

11.3.1.18.3 Field Testing

Test piles are employed at a project site for two purposes:

- For test driving, to determine the length of pile required prior to placing purchasing orders.
- For load testing, to verify actual pile capacity versus design capacity for nominal axial resistance.

11.3.1.18.3.1 Installation of Test Piles

Test piles are not required for spliceable types of piles. Previous experience indicates that contractors typically order total plan quantities for cast-in-place or steel H-piling in 60-foot lengths. The contractor uses one of the driven structure piles as a test pile at each designated location.

Test piling should be driven near the location of a soil boring where the soil characteristics are known and representative of the most unfavorable conditions at the site. The test pile must be exactly the same type and dimension as the piles to be used in the construction and installed by the same equipment and manner of driving. A penetration record is kept for every 1 foot of penetration for the entire length of pile. This record may be used as a guide for future pile driving on the project. Any subsequent pile encountering a smaller resistance is considered as having a smaller nominal resistance capacity than the test pile.

11.3.1.18.3.2 Static Pile Load Tests

A static pile load test is usually conducted to furnish information to the geotechnical engineer to develop design criteria or to obtain test data to substantiate nominal resistance capacity for piles. A static pile load test is the only reliable method of determining the nominal bearing resistance of a single pile, but it is expensive and can be quite time consuming. The decision to embark on an advance test program is based upon the scope of the project and the complexities of the foundation conditions. Such test programs on projects with large numbers of displacement piling often result in substantial savings in foundation costs, which can more than offset the test program cost. WisDOT has only performed a limited number of pile load tests on similar type projects.

Static pile load testing generally involves the application of a direct axial load to a single vertical pile. However, static pile load testing can involve uplift or axial tension tests, lateral tests applied horizontally, group tests or a combination of these applied to battered piles. Most static test loads are applied with hydraulic jacks reacting against either a stable loaded platform or a test frame anchored to reaction piles.

The basic information to be developed from the static pile load test is usually the deflection of the pile head under the test load. Movement of the head is caused by elastic deformation of the piles and the soil. Soil deformation may cause undue settlement and must be guarded against. The amount of deformation is the significant value to be obtained from load tests, rather than the total downward movement of the pile head. Static pile load tests are typically performed by loading to a given deflection value.

It is impractical to test every pile on a project. Therefore, test results can be applied to other piles or pile groups providing that the following conditions exist:

- The other piles are of the same type, material and size as the test piles.
- Subsoil conditions are comparable to those at the test pile locations.
- Installation methods and equipment used are the same as, or comparable to, those used for the test piles.
- Piles are driven to the same penetration depth or resistance or both as the test piles to compensate for variations in the vertical position and density of the bearing strata.

11.3.1.19 Construction Monitoring for Economic Evaluation of Deep Foundations

The goal of the foundation design is to provide the most efficient and economical design for the subsurface conditions. The design of pile-supported foundations is influenced by the resistance factor, which is generally a function of pile resistance determination during installation. The discussion in 11.3.1.14 presents the definition of resistance factors.

The typical method for a majority of the Department's deep foundation substructures is using the modified Gates to determine the RDR and to use a resistance factor of 0.50 based on department research and past experience. A comparison should be made between the use of the modified Gates and the use of the PDA with CAPWAP or the use of the Static Pile Load Test and the PDA with CAPWAP to determine which method is the most economical.

There are two possible methods available to economically use the PDA with CAPWAP to determine the required driving resistance, which allows the use of a resistance factor of 0.65.

Method 1: Reduce the number of piles in the substructure by driving the piles to the same RDR as using the modified Gates, but then increasing the FACR used in design. This is possible because the department has set a maximum value on the RDR, which when converted to the FACR is less than the structural capacity of the piles. This is true for all H-piles, and for some CIP piles when the FACR is controlled by the maximum allowable compression stress during driving based on 90 percent of the specified yield stress of steel.

Method 2: Drive each pile to a lower RDR, which should result in a shorter pile length. The number of piles per substructure would remain the same. The design estimated pile lengths are a function of the assumed soil conditions and the required driving resistance. The as-built pile lengths are a function of the actual soil conditions encountered and the contractor's hammer selection.

The department recommends Method 1 when evaluating the potential economic benefits of using the PDA with CAPWAP, because of the difficultly in accurately predicting pile lengths.

The method used to compare modified Gates to Static Pile Load Test(s) and the PDA with CAPWAP, which allows the use of a resistance factor of 0.80, would follow the procedures described in Method 1 used in the PDA with CAPWAP, reducing the number of piles per substructure. The number of static load test(s) will be a function of the size and number of substructures, the general spatial extent of the area in question and the variability of the subsurface conditions in the area of interest.

The costs to be included in the economic evaluation include the cost of the piling, the cost for the Department/Consultant to monitor the test piles, the cost for the Consultant CAPWAP evaluation (the Department does not currently have this capability), the unit costs for the contractor's time for driving and redriving the test piles, and the cost for the static pile load test(s).

Once the investigation of the subsurface conditions has been completed the geotechnical engineer and the structure engineer should discuss the potential for cost savings by increasing the resistance factor. The Bureau of Structures, Geotechnical Engineering Unit and the Region should be included in the discussion and should be part of the decision. Generally, the larger the project, the greater the potential for significant savings. The Department has two PDA's; therefore, the project team should contact the Geotechnical Engineering Unit to evaluate resources prior to incorporation of an increased resistance factor in the foundation design. PDA monitoring may be completed by Department or consultant personnel.

The following two examples use Method 1 to illustrate the potential cost savings/expenses for PDA with CAPWAP:

Pier
Pier Example: 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of $40/foot. (**Note:** It is realized that for pier design the number of piles is not exclusively related to the vertical load, but this example is simplified for illustrative purposes).
Modified Gates: RDR = 220 tons, FACR = 110 tons, Total Load on Pier = 3,500 tons, Number of Piles = 3,500 tons / 110 tons = 32 piles Pile Cost = 32 piles x 100 feet x $40/ft = $128,000 Total Cost = $128,000
PDA/CAPWAP: RDR = 220 tons, FACR = 143 tons, Total Load on Pier = 3,500 tons, Number of Piles = 3,500 tons / 143 tons = 25 piles Pile Cost = 25 piles x 100 feet x $40/ft = $100,000 PDA Testing Cost = 2 piles/sub. x $700/pile = $1,400 PDA Restrike Cost = 2 piles/sub. x $600/pile = $1,200 CAPWAP Evaluation = 1 eval./sub. x $400/eval. = $400 Total Cost = $103,000
PDA/CAPWAP Savings = $25,000/pier
Abutment
Abutment Example: 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of $40/foot.
Modified Gates: RDR = 220 tons, FACR = 110 tons, Total Load on Abut = 980 tons, Number of Piles = 980 tons / 110 tons = 9 piles Total Cost = 9 piles x 100 feet x $40/ft = $36,000
PDA/CAPWAP: RDR = 220 tons, FACR = 143 tons, Load on Abut = 980 tons, Number of Piles = 980 tons / 143 tons = 7 piles, however because of maximum spacing requirements the design will need 8 piles. Pile Cost = 8 piles x 100 feet x $40/ft = $32,000 PDA Testing Cost = 2 piles/sub. x $700/pile = $1,400 PDA Restrike Cost = 2 piles/sub. x $600/pile = $1,200 CAPWAP Evaluation = 1 eval./sub. x $400/eval. = $400

Total Cost = $35,000
PDA/CAPWAP Cost = $1000/abutment **Note:** For a three span bridge, with 12 x 53 H-piles to an estimated length of 100 feet at a unit cost of $40/foot, PDA/CAPWAP would provide an estimated structure savings of $52,000. For a two span bridge, with 12 x 53 H-piles to an estimated length of 40 feet at a unit cost of $40/foot, PDA/CAPWAP would provide an estimated structure savings of $5,400. Bid prices based on 2014-2015 cost data.

Table 11.3-6
Economical Evaluation for Deep Foundations with Two Construction Monitoring Methods

11.3.2 Drilled Shafts

11.3.2.1 General

Drilled shafts are generally large diameter, cast-in-place, open ended, cased concrete piles which are designed to carry extremely heavy loads. Drilled shafts can be the most economical foundation alternative at sites where foundation loads are carried to bearing on dense strata or bedrock. They are also cost effective in water crossings with very shallow bedrock, where cofferdams are difficult or expensive to construct, and where high overturning moments must be resisted.

Drilled shafts are installed by removing soil and rock using drilling methods or other excavation techniques and constructing the foundation element in the excavated hole. The excavated hole may be supported using temporary or permanent casing, drilling slurry or other methods. The hole is then filled with a reinforcement cage and cast-in-place concrete. Drilled shafts are non-displacement elements since the soil volume required for the element is physically removed prior to installation. Thus, the effective normal stress adjacent to the pile remains unchanged or is reduced (due to expansion of the soil into the hole before insertion/construction of the load bearing element), and the soil properties and pore water pressure adjacent to the foundation elements are not significantly impacted.

Because drilled shafts do not require a hammer for installation and do not displace the soil, they typically have much less impact on adjacent structures. Depending on the excavation technique used, they can penetrate significant obstructions. Because the method of construction often allows a decrease in the effective stress immediately adjacent to and beneath the tip of the foundation element, the resistance developed will often be less than an equivalently sized driven pile.

Drilled shafts are generally considered fixed to the substructure unit if the reinforcing steel from the shaft is fully developed within the substructure unit.

Drilled shafts vary in diameter from approximately 2.5 to 10 feet. Drilled shafts with diameters greater than 6 feet are generally referred to as piers. Shafts may be designed to transfer load to the bearing stratum through side friction, point-bearing or a combination of both. The drilled shaft may be cased or uncased, depending on the subsurface conditions and depth of bearing.

The minimum drilled shaft spacing shall be 3.0 shaft diameters center-to-center (3D). When drilled shafts are spaced less than 6D, group effects shall be evaluated for possible reductions to axial and lateral resistances. See 11.3.2.3.3 for more information.

Drilled shafts have been used on only a small number of structures in Wisconsin. For unusual site conditions, the use of drilled shafts may be advantageous. Design methodologies for drilled shafts can be found in **LRFD [10.8]** Drilled Shafts and *Drilled Shafts: Construction Procedures and Design Methods*. FHWA Publication NHI-18-024, FHWA GEC 010. 2018.

Strength limit states for drilled shafts are evaluated in the same way as for driven piles. Drivability is not required to be evaluated. The structural resistance of drilled shafts is evaluated in accordance with **LRFD [5.6 and 5.7]**. This includes evaluation of axial resistance, combined axial and flexure, shear and buckling. It is noted that the critical load case for combined axial and flexure may be a load case that results in the minimum axial load or tension.

11.3.2.2 Resistance Factors

Resistance factors for drilled shafts are presented in Table 11.3-7 and are selected based on the method used to determine the nominal (ultimate) resistance capacity of the drilled shaft. The design intent is to adjust the resistance factor based on the reliability of the method used to determine the nominal shaft resistance. As with driven piles, the selection of a geotechnical resistance factor should be based on the intended method of resistance verification in the field. Because of the cost and difficulty associated with testing drilled shafts, much more reliance is placed on static analysis methods.

Condition/Resistance Determination Method				Resistance Factor
Static Analysis – Used in Design Phase	Nominal Resistance of Single-Drilled Shaft in Axial Compression, φ_{stat}	Shaft Resistance in Clay	Alpha Method	0.45
		Point Resistance in Clay	Total Stress	0.40
		Shaft Resistance in Sand	Beta Method	0.55
		Point Resistance in Sand	O'Neill and Reese	0.50
		Shaft Resistance in IGMs	O'Neill and Reese	0.60
		Point Resistance in IGMs	O'Neill and Reese	0.55
		Shaft Resistance in Rock	Horvath and Kenney O'Neill and Reese	0.55
			Carter and Kulhawy	0.50
		Point Resistance in Rock	Canadian Geotech. Soc.	0.50

			Pressuremeter Method O'Neill and Reese	
	Block Failure, φ_{bl}	Clay		0.55
	Uplift Resistance of Single-Drilled Shaft, φ_{up}	Clay	Alpha Method	0.35
		Sand	Beta Method	0.45
		Rock	Horvath and Kenney Carter and Kulhawy	0.40
	Group Uplift Resistance, φ_{ug}	Sand and Clay		0.45
	Horizontal Geotechnical Resistance of Single Shaft or Pile Group	All Soil Types and Rock		1.0

Table 11.3-7

Geotechnical Resistance Factors for Drilled Shafts **LRFD [Table 10.5.5.2.4-1]**

For drilled shafts, the base geotechnical resistance factors in Table 11.3-7 assume groups containing two to four shafts, which are slightly redundant. For groups containing at least five elements, the base geotechnical resistance factors in Table 11.3-7 should be increased by 20%.

WisDOT policy item:

When a bent contains at least 5 columns (where each column is supported on a single drilled shaft) the resistance factors in Table 11.3-7 should be increased up to 20 percent for the Strength Limit State.

For piers supported on a single drilled shaft, the resistance factors in Table 11.3-7 should be decreased by 20 percent for the Strength Limit State. Use of single drilled shaft piers requires approval from the Bureau of Structures.

Resistance factors for structural design of drilled shafts are obtained from **LRFD [5.5.4.2]**.

11.3.2.3 Bearing Resistance

Most drilled shafts provide geotechnical resistance in both end bearing and side friction. Because the rate at which side friction mobilizes is usually much higher than the rate at which end bearing mobilizes, past design practice has been to ignore either end bearing for shafts with significant sockets into the bearing stratum or to ignore skin friction for shafts that do not penetrate significantly into the bearing stratum. This makes evaluation of the geotechnical resistance slightly more complex, because in most cases it is not suitable to simply add the nominal (ultimate) end bearing resistance and the nominal side friction resistance in order to obtain the nominal axial geotechnical resistance.

When computing the nominal geotechnical resistance, consideration must be given to the anticipated construction technique and the level of construction control. If it is anticipated to be difficult to adequately clean out the bottom of the shafts due to the construction technique or subsurface conditions, the end bearing resistance may not be mobilized until very large deflections have occurred. Similarly, if construction techniques or subsurface conditions result in shaft walls that are very smooth or smeared with drill cuttings, side friction may be far less than anticipated.

Because these resistances mobilize at different rates, it may be more appropriate to add the ultimate end bearing to that portion of the side resistance remaining at the end of bearing failure. Or it may be more appropriate to add the ultimate side resistance to that portion of the end bearing mobilized at side resistance failure. Note that consideration of deflection, which is a service limit state, may control over the axial geotechnical resistance since displacements required to mobilize the ultimate end bearing can be excessive.

11.3.2.3.1 Shaft Resistance

The shaft resistance is estimated by summing the friction developed in each stratum. When drilled shafts are socketed in rock, the shaft resistance that is developed in soil is generally ignored to satisfy strain compatibility. The following analysis methods are typically used to compute the static shaft resistance in soil and rock:

- Alpha method for cohesive soil, as specified in **LRFD [10.8.3.5.1]**
- Beta method (β-method) for cohesionless soil, as specified in **LRFD [10.8.3.5.2]**
- Horvath and Kenny method for rock, as specified in **LRFD [10.8.3.5.4]**

11.3.2.3.2 Point Resistance

The following analysis methods are typically used to compute the static shaft resistance in soil:

- Alpha method for cohesive soil, as specified in **LRFD [10.8.3.5.1]**
- Beta method (β-method) for cohesionless soil, as specified in **LRFD [10.8.3.5.2]**

The ultimate unit point resistance of a drilled shaft in intact or tightly jointed rock is computed as 2.5 times the unconfined compressive strength of the rock. For rock containing open or filled joints, the geomechanics RMR system is used to characterize the rock, and the ultimate point resistance in rock can be computed as specified in **LRFD [10.8.3.5.4c]**.

11.3.2.3.3 Group Capacity

Group effects for axial and lateral resistances shall be evaluated in accordance with **LRFD [10.8.3.6]** and **LRFD [10.8.3.8]**, respectively. In general, reductions to individual nominal resistances are limited to drilled shafts spaced less than 6D and are based on spacing, soil type, and soil contact.

11.3.2.4 Lateral Load Resistance

Because drilled shafts are made of reinforced concrete, the lateral analysis should consider the nonlinear variation of bending stiffness with respect to applied bending moment. At small applied moments, the reinforced concrete section performs elastically based on the size of the section and the modulus of elasticity of the concrete. At larger moments, the concrete cracks in tension and the stiffness drops significantly.

11.3.2.5 Other Considerations

Detailing of the reinforcing steel in a drilled shaft must consider the constructability of the shaft. The reinforcing cages must be stiff enough to resist bending during handling and concrete placement. In addition, the spaces between reinforcement bars must be kept large enough to permit easy flow of the concrete from the center of shaft to the outside of shaft. These two requirements will generally force the use of larger, more widely spaced longitudinal and transverse reinforcement bars than would be used in the design of an above-grade column. In addition, when using hooked bars to tie the shaft to the foundation, consideration must also be given to concrete placement requirements and temporary casing removal requirements.

11.3.3 Micropiles

11.3.3.1 General

In areas of restricted access, close proximity to settlement sensitive existing structures or difficult geology, micropiles may be considered when determining the recommended foundation type. Although typically more expensive than driven pile, constructability considerations may warrant selection of micropiles as the preferred foundation type. A micropile is constructed by drilling a borehole with drill casing, placing reinforcement and grouting the hole. Micropiles are installed by methods that cause minimal disturbance to adjacent structures, soil and the environment. They can be installed in areas with restricted access and vertical clearance. Drill casing permits installation in poor ground conditions. Micropiles are installed with the same type of equipment that is used for ground anchor and grouting projects. Micropiles can be either vertical or battered.

Micropiles are used for structural support of new structures, underpinning existing structures, scour protection and seismic retrofit at existing structures. Micropiles are also used to create a reinforced soil mass for ground stabilization.

With a micropile’s smaller cross-sectional area, the pile design is more frequently governed by structural and stiffness considerations. Due to the small pile diameter, point resistance is usually disregarded for design. Steel casing for micropiles is commonly delivered in 5 to 20 foot long flush-joint threaded sections. The casing is typically 5.5 to 12 inches in diameter, with yield strength of 80 ksi. Grout is mixed neat with a water/cement ratio on the order of 0.45 and an unconfined compressive strength of 4 to 6 ksi. Grade 60, 90 and 150 single reinforcement bars are generally used with centralizers.

Grout/ground bond capacity varies directly with the method of placement and pressure used to place the grout. Common methods include grout placement under gravity head, grout placement under low pressure as temporary drill steel is removed and grout placement under high pressure using a packer and regrout tube. Some regrout tubes are equipped to allow regrouting multiple times to increase pile capacity.

11.3.3.2 Design Guidance

Micropiles shall be designed in conformance with the current *AASHTO LRFD* and in accordance with the WisDOT Bridge Manual. Design guidelines for micropiles are provided in FHWA Publication No. FHWA-NHI-05-039.

11.3.4 Augered Cast-In-Place Piles

11.3.4.1 General

Augered cast-in-place (ACIP) piles are installed by drilling a hole with a hollow stem auger. When the auger reaches a design depth (elevation) or given torque, sand-cement grout or concrete is pumped through the hollow-stem auger while the auger is withdrawn from the ground. Reinforcement steel can be placed while the grout is still fluid. A single reinforcement bar can also be installed inside the hollow stem auger before the auger is extracted. ACIP piles are installed by methods that cause minimal disturbance to adjacent structures, soil and the environment. They can also be installed in areas with restricted access and vertical clearance. Temporary casing is not required. In many situations, these foundation systems can be constructed more quickly and less expensively than other deep foundation alternatives.

ACIP piles are generally available in 12- to 36-inch diameters and typically extend to depths of 60 to 70 feet. In some cases, ACIP piles have been installed to depths of more than 100 feet. The torque capacity of the drilling equipment may limit the available penetration depth of ACIP piles, especially in stiff to hard cohesive soil. Typical Wisconsin bridge contractors do not own the necessary equipment to install this type of pile.

ACIP piles may be more economical; however, there is a greater inherent risk in their installation from the quality control standpoint. There is currently no method available to determine pile capacity during construction of ACIP piles. WisDOT does not generally use this pile type unless there are very unusual design/site requirements.

11.3.4.2 Design Guidance

In the future, the FHWA will distribute a Geotechnical Engineering Circular that will provide design and construction guidance for ACIP piles. WisDOT plans to reassess the use of ACIP piles at that time.

11.4 References

1. State of Wisconsin, Department of Transportation. *Standard Specifications for Highway and Structure Construction*.

2. American Association of State Highway and Transportation Officials. *Standard Specifications for Highway Bridges*.

3. American Association of State Highway and Transportation Officials. *AASHTO LRFD Bridge Design Specifications*.

4. Armour, T., P. Groneck, J. Keeley and S. Sharma. *Micropile Design and Construction Guidelines, Implementation Manual*. FHWA Publication SA-97-070. June 2000.

5. Chellis, R.D. *Pile Foundations*. McGraw Hill. 1961.

6. Hannigan, P.J., G.G. Goble, G. E. Likins and F. Rausche. *Design and Construction of Driven Pile Foundations – Volumes 1 and 2*. FHWA Publication NHI-05-042. 2006.

7. Kimmerling, R.E., R.C. Bachus, P.W. Mayne, J.A. Scheider and T.E. Zettler. *Geotechnical Engineering Circular No. 6: Shallow Foundations*. FHWA Publication SA-02-054. September 2002.

8. Lagrasse, P.F., J.D. Schall and E.V. Richardson. *Hydraulic Engineering Circular No. 20: Stream Stability at Highway Structures*, 3rd Edition. FHWA Publication NHI 01-002. March 2001.

9. Lagrasse, P.F., L.W. Zevenbergen, J.D. Schall and P.E. Clopper. *Hydraulic Engineering Circular No. 23: Bridge Scour and Stream Instability Countermeasures, Experience, Selection, and Design Guidance*, 2nd Edition. FHWA Publication NHI 01-003. March 2001.

10. Leonards, G.A. *Foundation Engineering*. McGraw Hill. 1962.

11. Michael Baker Jr., Inc. *LRFD for Highway Bridge Substructures and Earth Retaining Structures*. FHWA Publication NHI-05-095. Revised October 2006.

12. *Micropile Design and Construction Reference Manual*. FHWA Publication NHI-05-039. 2005.

13. O'Neil, M.W. and L.C. Reese. *Drilled Shafts: Construction Procedures and Design Methods*. FHWA Publication IF-99-025. August 1999.

14. O'Neil, M.W., F.C. Townsend, K.M. Hassan, A. Butler and P.S. Chan. *Load Transfer for Drilled Shafts in Intermediate Geomaterials*. FHWA Publication RD-95-172. November 1996.

15. Richardson, E.V. and S.R. Davis. *Hydraulic Engineering Circular No. 18: Evaluating Scour at Bridges*, 4th Edition. FHWA Publication NHI 01-001. May 2001.

16. Sabatini, P.J. et al. *Geotechnical Engineering Circular No. 5: Evaluation of Soil and Rock Properties*. FHWA Publication IF-02-034. April 2002.

17. Terzaghi, K. and R.B. Peck. *Soil Mechanics in Engineering Practice*, 2nd Edition. Wiley. 1967.

18. Tschebotrarioff, G.P. *Foundations, Retaining and Earth Structures*, 2nd Edition. McGraw Hill. 1973.

19. *Drilled Shafts: Construction Procedures and Design Methods*. FHWA Publication NHI-18-024, FHWA GEC 010. 2018.

11.5 Design Examples

WisDOT will provide design examples.

This section will be expanded later when the design examples are available.

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Table of Contents

- *Wall Gabion, Item* SPV.0165*

** Contact BOS Design before using and to obtain the most recent version.*

Note that the use of QMP Special Provisions began with the December 2014 letting or prior to December 2014 letting at the Region's request. Special provisions are available on the Wisconsin Bridge Manual website.

The designer determines what wall systems(s) are applicable for the project. The approved names of suppliers are inserted for each eligible wall system. The list of approved proprietary wall suppliers is maintained by the Bureau of Structures which is responsible for the Approval Process for earth retaining walls, 14.16.

14.16 Submittal Requirements for Pre-Approval Process

14.16.1 General

The following four wall systems require the supplier or manufacturer to submit to the Structural Design Section a package that addresses the items specified in 14.16.3.

1. Modular Block Gravity Walls
2. MSE Walls with Modular Block Facings
3. MSE Walls with Precast Concrete Panel Facings
4. Modular Concrete Bin or Crib Walls

14.16.2 General Requirements

Approval of retaining wall systems allows for use of these systems on Wisconsin Department of Transportation (WisDOT) projects upon the manufacturer's certification that the system as furnished to the contractor (or purchasing agency) complies with the design procedures specified in the *Bridge Manual*. WisDOT projects include: State, County and Municipal Federal Aid and authorized County and Municipal State Aid projects in addition to materials purchased directly by the state.

The manufacturer shall perform all specification tests with qualified personnel and maintain an acceptable quality control program. The manufacturer shall maintain records of all its control testing performed in the production of retaining wall systems. These test records shall be available at all times for examination by the Construction Materials Engineer for Highways or designee. Approval of materials will be contingent upon satisfactory compliance with procedures and material conformance to requirements as verified by source and field samples. Sampling will be performed by personnel during the manufacture of project specific materials.

14.16.3 Qualifying Data Required For Approval

Applicants requesting Approval for a specific system shall provide three copies of the documentation showing that they comply with *AASHTO LRFD* and *WisDOT Standard Specifications* and the design criteria specified in the *Bridge Manual*.

1. An overview of the system, including system theory.
2. Laboratory and field data supporting the theory.
3. Detailed design procedures, including sample calculations for installations with no surcharge, level surcharge and sloping surcharge.
4. Details of wall elements, analysis of structural elements, capacity demand ratio, load and resistance factors, estimated life, corrosion design procedure for soil reinforcement elements, procedures for field and laboratory evaluation including instrumentation and special requirements, if any.

Table of Contents

30.1 Crash-Tested Bridge Railings and FHWA Policy

Notice: All contracts with a letting date after December 31, 2019 must use bridge rails and transitions meeting the 2016 Edition of MASH criteria for new permanent installation and full replacement.

WisDOT policy item:

For all Interstate structures, the 42SS parapet shall be used. For all STH and USH structures with a posted speed >= 45 mph, the 42SS parapet shall be used.

The timeline for implementation of the above policy is:

- All contracts with a letting date after December 31, 2019.
  (This is an absolute, regardless of when the design was started.)
- All preliminary designs starting after October 1, 2017
  (Even if the let is anticipated to be prior to December 31, 2019.)

Contact BOS should the 42” height adversely affect sight distance, a minimum 0.5% grade for drainage cannot be achieved, or for other non-typical situations.

Crash test procedures for full-scale testing of guardrails were first published in 1962 in the *Highway Research Correlation Services Circular 482*. This was a one-page document that specified vehicle mass, impact speed, and approach angle for crash testing and was aimed at creating uniformity to traffic barrier research between several national research agencies.

In 1974, the National Cooperative Highway Research Program (NCHRP) published their final report based on NCHRP Project 22-2, which was initiated to address outstanding questions that were not covered in *Circular 482*. The final report, NCHRP Report 153 – *“Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances,”* was widely accepted following publication; however, it was recognized that periodic updating would be required.

NCHRP Project 22-2(4) was initiated in 1979 to address major changes to reflect current technologies of that time and NCHRP Report 230, *“Recommended Procedures for the Safety Performance Evaluation of Highway Safety Appurtenances,”* was published in 1980. This document became the primary reference for full-scale crash testing of highway safety appurtenances in the U.S. through 1993.

In 1986, the Federal Highway Administration (FHWA) issued a policy memorandum that stated highway bridges on the National Highway System (NHS) and the Interstate Highway System (IHS) must use crash-tested railings in order to receive federal funding.

The American Association of State Highway and Transportation Officials (AASHTO) recognized that the evolution of roadside safety concepts, technology, and practices necessitated an update to NCHRP Report 230 approximately 7 years after its adoption. NCHRP Report 350, *“Recommended Procedures for the Safety Performance Evaluation of Highway Features,”* represented a major update to the previously adopted report. The updates

8. Chain Link Fence and Tubular Screening, as shown in the Standard Details, may be attached to the top of concrete parapets as part of a Combination Railing or as a Pedestrian Railing attached directly to the deck if protected by a Traffic Railing between the roadway and a sidewalk at grade. Due to snagging and breakaway potential of the vertical spindles, top-mounted Tubular Screening and Chain Link Fence should only be used when the design speed is 45 mph or less, or the screening is protected by a Traffic Railing between the roadway and a sidewalk at grade.

   Contact the Bureau of Structures Development Section when protective screening is warranted and used for design speeds exceeding 45 mph. In some cases, a Chain Link Fence mounted on the outside face (side-mounted) of the concrete parapet may be acceptable.

9. Type "H" aluminum or steel railing can be used on top of either vertical face or single slope parapets (“A” or “SS”) as part of a Combination Railing when required for pedestrians and/or bicyclists. For a design speed greater than 45 mph, the single slope parapet is recommended. Per the Standard Specifications, the contractor shall furnish either aluminum railing or steel railing. In general, the bridge plans shall include both options. For a specific project, one option may be required. This may occur when rehabilitating a railing to match an existing railing or when painting of the railing is required (requires steel option). If one option is required, the designer shall place the following note on the railing detail sheet: “Type H (insert railing type) railing shall not be used”. The combination railing is TL-3 under MASH.

10. Timber Railing as shown in the Standard Details is not allowed on the National Highway System (NHS). Timber Railing may be used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less. The Timber Railing has not been rated under MASH.

11. The type "W" railing, as shown in the Standard Details, is not allowed on the National Highway System (NHS). This railing may be used as a Traffic Railing on non-NHS roadways with a design speed of 45 mph or less. The type “W” railing shall be used on concrete slab structures only. The use of this railing on girder type structures has been discontinued. Generally, type "W" railing is considered when the roadway approach requires standard beam guard and if the structure is 80 feet or less in length. Although the type “W” railing was crash-tested per NCHRP Report 230 and meets NCHRP Report 350 crash test criteria for TL-3 (based on a May 1997 FHWA memorandum), FHWA has since restricted its use as indicated above.

12. Type “M” steel railing, as shown in the Standard Details, shall generally be used as a Traffic Railing on all functional classes of Wisconsin structures with a design speed of 45 mph or less. The type “M” railing may be used on roadways with a design speed exceeding 45 mph where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints. The type “M” railing also can be used in place of the type “W” railing when placed on girder type structures as type “W” railings are not allowed for this application. However, the type “M” railing is not allowed for use on prestressed box girder bridges. This railing shall be considered where the Region requests an open railing. The type “M” railing is TL-2 under MASH.

13. Type “NY3/NY4” steel railings, as shown in the Standard Details, shall generally be used as a Traffic Railing on all functional classes of Wisconsin structures with a design speed of 45 mph or less. The type “NY3/NY4” railings may be used on roadways with a design speed exceeding 45 mph where the minimum 0.5% deck grade cannot be accommodated for proper drainage based on project specific constraints. The type “NY3/NY4” railings also can be used in place of the type “W” railing when placed on girder type structures as type “W” railings are not allowed for this application. The type “NY4” railing may be used on a raised sidewalk where the design speed is 45 mph or less. However, the type “NY” railings are not allowed for use on prestressed box girder bridges. These railings shall be considered where the Region requests an open railing. The type “NY” railings are TL-2 under MASH.

14. The type "F" steel railing, as shown in the Standard Details of Chapter 40, shall not be used on new bridge plans with a PS&E after 2013. It has not been allowed on the National Highway System (NHS) in the past and was used on non-NHS roadways with a design speed of 45 mph or less. Details in Chapter 40 are for informational purposes only.

15. If a box culvert has a Traffic Railing across the structure, then the railing members shall have provisions for a thrie beam connection at the ends of the structure as shown in the Facilities Development Manual (FDM) SDD 14b20. Railing is not required on box culverts if the culvert header is located outside of the clear zone. Non-traversable hazards or fixed objects should not be constructed or allowed to remain within the clear zone. When this is not feasible, the use of a Traffic Railing to shield the hazard or obstacle may be warranted. Use of traversable grates within the clear zone has been limited to small drainage applications due to clogging concerns and are subject to prior approval by the Bureau of Structures. Refer to FDM 11-45 for additional information on drainage end treatments.

16. When the structure approach thrie beam is extended across the box culvert; refer to Standard Detail, Guardrail Post Anchorage System for additional information. The minimum dimension between end of box and face of guard rail provides an acceptable rail deflection to prevent a vehicle wheel from traversing over the end of the box culvert. In almost every case, the timber posts with offset blocks and standard beam guard are used. Type "W" railing may be used for maintenance and box culvert extensions to mitigate the effect of structure modifications.

See the FDM for additional railing application requirements. See FDM 11-45 for Traffic Barrier, Crash Cushions, and Roadside Barrier Design Guidance. See FDM 11-35-1 Table 1.2 for requirements when barrier wall separation between roadway and sidewalk is necessary.

Table of Contents

36.1 Design Method

36.1.1 Design Requirements

All new box culverts are to be designed using *AASHTO LRFD Bridge Design Specifications*, hereafter referred to as *AASHTO LRFD*.

36.1.2 Rating Requirements

The current version of *AASHTO Manual for Bridge Evaluation (LRFR)* covers rating of concrete box culverts. Refer to 45.8 for additional guidance on load rating various types of culverts.

36.1.3 Standard Permit Design Check

New structures are also to be checked for strength for the 190 kip Wisconsin Standard Permit Vehicle (Wis-SPV), with a single lane loaded, multiple presence factor equal to 1.0, and a live load factor (γ_{LL}) as shown in Table 45.3-3. See 45.12 for the configuration of the Wis-SPV. The structure should have a minimum capacity to carry a gross vehicle load of 190 kips, while also supporting the future wearing surface (where applicable – future wearing surface loads are only applied to box culverts with no fill). When applicable, this truck will be designated as a Single Trip Permit Vehicle. It will have no escorts restricting the presence of other traffic on the culvert, no lane position restrictions imposed and no restrictions on speed to reduce the dynamic load allowance, IM. The maximum Wisconsin Standard Permit Vehicle load that the structure can resist, calculated including current wearing surface loads, is shown on the plans.

36.5 Design Information

Sidesway of the box is not considered because of the lateral support of the soil.

The centerline of the walls and top and bottom slabs are used for computing section properties and dimensions for analysis.

WisDOT Policy Item:

For skews 20 degrees or less, place the reinforcing steel along the skew. For skews over 20 degrees, place the reinforcing steel perpendicular to the centerline of box.

Culverts are analyzed as if the reinforcing steel is perpendicular to the centerline of box for all skew angles.

The minimum thickness of the top and bottom slab is 6½ inches. For pedestrian underpasses and slabs with fills less than 2 feet, the minimum thickness of the top slab should be 1 foot. Minimum wall thickness is based on the inside opening of the box (height) and the height of the apron wall above the floor. Use the following table to select the minimum wall thickness that meets or exceeds the three criteria in the table.

Minimum Wall Thickness (Inches)	Cell Height (Feet)	Apron Wall Height Above Floor (Feet)
8	< 6	< 6.75
9	6 to < 10	6.75 to < 10
10	10 to > 10	10 to < 11.75
11		11.75 to < 12.5
12		12.5 to 13

Table 36.5-1
Minimum Wall Thickness Criteria

All slab thicknesses are rounded to the next largest ½ inch.

Top and bottom slab thicknesses are determined by shear and moment requirements. Slab thickness shall be adequate to carry the factored shear without shear reinforcement.

All bar steel is detailed as being 2 inches clear with the following exceptions:

- The bottom steel in the bottom slab is detailed with 3 inches clear
- The top steel in the top slab of a box culvert with no fill is detailed with 2½ inches clear

A haunch is provided only when the slab depth required at the interior wall is more than 2 inches greater than that required for the remainder of the span. Only 45º haunches shall be used. Minimum haunch depth and length is 6 inches. Haunch dimensions are increased in 3 inch increments.

The slab thickness required is determined by moment or shear, whichever governs.

The shear in the top and bottom slabs is assumed to occur at a distance "d" from the face of the walls. The value for "d" equals the distance from the centroid of the reinforcing steel to the face of the concrete in compression. When a haunch is used, shear must also be checked at the end of the haunch.

For multi-cell culverts make interior and exterior walls of equal thickness.

Culverts shall be designed for live load and the range of fill between the shoulders of the roadway. The depth of fill is measured from the top of culvert to the surface of earth fill or top of roadway pavement. To accommodate future widening of the roadway, reduced sections may not be used on the ends of the culvert where there is less fill. Exceptions may be made with the approval of the Bureau of Structures where the culvert has high fills and a reduced section could be used for at least two panel pours per end of culvert. Culvert extensions shall be designed for the same range of fills as the original culvert. The extension design shall not have lower capacity than the original culvert. Maximum length of panel pour is 40 feet.

Culvert barrel lengths are typically determined by locating culvert headers outside of the clear zone. When extending headers beyond the clear zone is not practical, headers should be shielded by a suitable barrier system. Culvert wing lengths are determined based on a minimum 2 1/2:1 slope of fill from the apron to top of box. Consideration shall be given to match the typical roadway cross slope. Refer to Chapter 30 for additional information on barriers systems.

Dimensions on drawings are given to the nearest ¼ inch only.

Table of Contents

40.1 General

New bridges are designed for a minimally expected life of 75 years. Preliminary design considerations are site conditions, structure type, geometrics, and safety. Refer to Bridge Manual Chapters 9 and 17 for Materials and Superstructure considerations, respectively. Comprehensive specifications and controlled construction inspection are paramount to obtaining high quality structures. Case history studies show that adequately consolidated and properly cured concrete with low water-cement ratios and good air void systems have lower absorption rates and provide greater resistance to scaling and chloride penetration under heavy traffic and exposure to de-icing chemicals. Applying protective surface treatments to new decks improves their resistance to first year applications of de-icing chemicals.

Most interstate and freeway structures are not subject to normal conditions and traffic volumes. Under normal environmental conditions and traffic volumes, original bridge decks have an expected life of 40 years. Deck deterioration is related to the deck environment which is usually more severe than for any of the other bridge elements. Decks are subjected to the direct effects of weather, the application of chemicals and/or abrasives, and the impact of vehicular traffic. For unprotected bar steel, de-icing chemicals are the primary cause of accelerated bridge deck deterioration. Chlorides cause the steel to corrode and the corrosion expansion causes concrete to crack along the plane of the top steel. Traffic breaks up the delaminated concrete leaving potholes on the deck surfaces. In general, deck rehabilitation on Wisconsin bridges has occurred after 15 to 22 years of service due to abnormally high traffic volumes and severe environment.

Full depth transverse floor cracks and longitudinal construction joints leak salt water on the girders below causing deterioration and over time, section loss.

Leaking expansion joints allow salt water seepage which causes deterioration of girder ends and steel bearings located under them. Also, concrete bridge seats will be affected in time. Concrete bridge seats should be finished flat, and sealed with a penetrating epoxy coating.

Bridges being designed with staged construction, whether new or rehabilitation, shall satisfy the requirements of LRFD (or LFD, if applicable) for each construction stage. Utilize the same load factors, resistance factors, load combinations, etc. as required for the final configuration, unless approved by Chief Structures Development Engineer at WisDOT.

40.13 Survey Report and Miscellaneous Items

Prior to scheduling bridge plans preparation, a Rehabilitation Structure Survey Report Form is to be completed for the proposed work with field and historical information. The Rehabilitation Structure Survey Report provides the Bridge Engineer pertinent information such as location, recommended work to be performed, and field information needed to accomplish the rehabilitation plans. A brief history of bridge construction date, type of structure, repair description and dates are also useful in making decisions as to the most cost effective rehabilitation. Along with this information, it is necessary to know the extent of deck delamination and current deck, super and substructure condition ratings. A thorough report recommending structural repairs and rehabilitation with corresponding notes is very useful for the designer including information from special inspections.

Because new work is tied to existing work, it is very important that accurate elevations be used on structure rehabilitation plans involving new decks and widenings. Survey information of existing beam seats is essential. Do not rely on original plans or a plan note indicating that it is the responsibility of the contractor to verify elevations.

Experience indicates that new decks open to traffic and subjected to application of de-icing salts within the first year show signs of early deck deterioration. Therefore, a protective surface treatment is applied to all new bridge deck concrete as given in the standard specifications.

For existing sloped faced parapets on decks under rehabilitation, the metal railings may be removed. On deck replacements and widenings, all existing railings are replaced. Refer to Bridge Manual Chapter 30-Railings for recommended railings.

On rehabilitation plans requiring removal of the existing bridge name plates; provide details on the plans for new replacement name plates if the existing name plate cannot be reused. The existing bridge number shall be used on the name plates for bridge rehabilitation projects.

If floor drain removal is recommended, review these recommendations for conformance to current design standards and remove any floor drains not required. Review each structure site for length of structure, grade, and water erosion at the abutments.

Bridge rehabilitation plans for steel structures are to provide existing flange and web sizes to facilitate selecting the proper length bolts for connections. Do not specify any aluminized paint that will come in contact with fresh concrete. The aluminum reacts severely with the fresh concrete producing concrete volcanoes.

Recommended paint maintenance is determined with assistance from the Wisconsin Structures Asset Management System (WiSAMS), which utilizes information provided by the routine bridge inspections.

Structure plans (using a sheet border with a #8 tab) are required for all structure rehabilitation projects. This includes work such as superstructure painting projects and all overlay projects, including polymer overlay projects. See Chapter 6-Plan Preparation guidance for plan minimum requirements.

Existing steel expansion devices shall be modified or replaced with watertight expansion devices as shown in Bridge Manual Chapter 28-Expansion Devices. If the hinge is repaired, consideration should be given to replacing the pin plates with the pins. Replace all pins with stainless steel pins conforming to ASTM 276, Type S20161 or equal. On unpainted steel bridges, the end 6’ or girder depth, whichever is greater, of the steel members adjacent to an expansion joint and/or hinge are required to have two shop coats of paint. The second coat is to be a brown color similar to rusted steel. Exterior girder faces are not painted for aesthetic reasons, but paint the hanger on the side next to the web.

A bridge on a steep vertical grade may slide downhill closing any expansion joint on this end and moving the girders off center from the bearings. One possible corrective action is to block the ends of the girder during the winter when the girders have shortened due to cold temperatures. Continued blocking over a few seasons should return the girders to the correct position. The expansion device at the upslope end may have to be increased in size.

Raised pavement markers require embedment in the travel way surface. As a general rule, they are not to be installed on bridge decks due to future leakage problems if the marker comes out. They shall not be installed thru the membranes on asphalt overlays and not at all on asphaltic decks where the waterproofing is integral with the surface.

Table of Contents

41.3.2.2 Priority Review

BOS provides a prioritized list of eligible structures work. Priority is determined using a priority index (PI); an algorithm developed by BOS. The algorithm considers data such as ADT, functional class, etc. This is intended to assist the regions as they program projects.

The Region may see fit to adjust the prioritized list based on regional system and operational factors.

41.3.2.3 Creating Improvement Projects with Structures Work Concepts

The next step in the programming process is for Regional Programming to develop structures improvement projects based on the list of individual structures work concepts. Projects may combine structures work as appropriate, but also consider pavement needs, safety needs, operational needs, etc.

There may be non-structural rationale for deviations from BOS-recommended scope and/or timing. Common reasons include, but are not limited to:

- Coordination with other improvement work (pavements, safety, operations, etc.)
- Traffic control costs
- User delay

If reasons such as those noted above are used to justify deviations from BOS-recommended scope and/or timing, a cost-benefit analysis should be performed to support the decision. More information on cost-benefit analysis and structures programming policy can be found in 41.6.6.

During this phase as projects are developed and up until the Structures Project Certification Phase (See 41.3.3), BOS asset management engineers will evaluate proposed projects on a regular basis to ensure that programmed structures work is eligible in terms of both scope and timing. Projects that contain only eligible structures work concepts or have appropriate justification for any deviations are considered *pre-certified*.

Only eligible projects or projects with appropriate justification will be considered for funding.

41.3.3 Structures Project Certification Phase (PY6-PY5, Life Cycle 10/11)

Structures project certification refers to the work required to produce the Bureau of Structures Certification Document (BOSCD). The components of the BOSCD are outlined in 41.3.3.6 below.

WisDOT policy item:

Any improvement project with state-owned B-Structure work (primary or secondary work concepts) requires certification.

41.3.3.1 BOS Structures Certification Liaison

BOS will designate a certification liaison for every structures improvement project, regardless of whether the project is designed by BOS or a consultant. The certification liaison will perform all of the work necessary for structures certification. A certification liaison will remain with each structures project (BOS-designed or consultant-designed) through the letting of that project, though the actual person assigned to a project may change over the lifecycle of that project.

41.3.3.2 Review of Primary Structures Work Concepts

Structures certification serves as the final review and approval for the scope and timing of the primary structures work concept. Regional planning engineers should only be selecting eligible structures work (scope and timing) for inclusion in transportation improvement projects. Additionally, BOS asset management engineers will evaluate projects on a regular basis (see 41.3.2) to ensure eligibility. With this process in place, the certification liaison will collaborate with BOS asset management engineers and Regional programming engineers (as necessary) to confirm scope and timing for primary structures work concepts.

41.3.3.3 Development of Secondary Structures Work Concepts

A key portion of the BOSCD is the early identification of secondary structures improvement work. Some examples of secondary work include, but are not limited to:

- Bearing rehabilitation or replacement
- Parapet or railing repairs
- Backwall or wingwall repairs
- Identification of specific substructure repairs
- Scour mitigation

Some items such as those above may have already been identified during the scoping of the primary structures work concepts. The certification liaison will review the existing inspection reports on file and consult the appropriate BOS inspection and maintenance personnel to identify any and all eligible secondary structures work concepts.

41.3.3.4 Development of the Structures Cost Estimate

A high-level cost estimate will have been developed as a part of the primary structures work concept. This estimate is for structures work only; costs for traffic control and mobilization are not included. The certification liaison will refine that estimate, taking into account the identified secondary structures improvement work. This estimate is not intended to be a final structures construction cost estimate, but is a refinement of the unit cost estimate previously developed.

41.3.3.5 Determination of Design Resourcing

As part of the structure's certification process, BOS will determine design resourcing and estimate the level of effort (in staff-hours) for the structures work. If BOS chooses to decline structures design for a given project, regional PDS staff should work with BOS consultant review supervisor to ensure selection of an appropriate consultant engineer for the project.

41.3.3.6 Bureau of Structures Certification Document (BOSCD)

The BOSCD includes information on all the items noted above, in addition to other key information identified by Region personnel. Additional project information and decision documentation can be found in the SCT.

41.3.4 Project Delivery and Execution Phase (PY4-Construction, Life Cycle 12+)

41.3.4.1 Structures Re-Certification

Any and all changes related to structures improvement work affecting items approved as part of the structures project certification shall be reviewed and approved by the certification liaison. This includes, but is not limited to, any of the following items:

- Scope (primary or secondary)
- Structures construction cost estimate
- PS&E or let date
- Advanceable date
- Structures design resourcing

The certification liaison for the project should be notified of any changes as soon as reasonably possible to approve/re-certify the project in a timely manner and not delay project schedule.

41.4 Structures Programming Process (Local System)

In general, local entities that own transportation structures may expend resources to preserve, rehabilitate, and replace structures at the owner's discretion. The state does require minimum information regarding all structures utilized for public transportation, and should be informed of structure work affecting the performance and/or capacity of the structure.

Local structure work may also be funded through the Local Bridge Assistance Program (Local Program). To support this program, BOS provides a prioritized eligibility list of bridge work concepts for the Division of Transportation Investment Management (DTIM), which is then posted publicly for the local owners. Local owners use the eligibility list to select projects for submission to the local program, and DTIM programs structure work on a biannual basis. Submission of eligible bridge work does not guarantee an entitlement of funds. According to Trans 213.03(4)(e), applications must both be approved and prioritized before determining entitlement of funds.

Not all bridges will have a work concept listed in the eligible bridge list. If an applicant believes work is necessary for a bridge that does not have a proposed work concept, or if the applicant believes a different work concept than the proposed work concept is more appropriate, the applicant can submit an alternate work concept. This will require an engineering report attached to the application for funding which describes the work concept proposed to be done, justification for the new work concept, and a life cycle cost analysis of different alternatives. Additional program requirements may apply.

More information about the Local Bridge Assistance Program can be found at the following link:
https://wisconsindot.gov/Pages/doing-bus/local-gov/astnce-pgms/highway/localbridge.aspx

41.4.1 Eligible Project Scopes

The structure work described in this section applies to eligible project scopes submitted in applications to the Local Bridge Assistance Program. These structure work scopes can be submitted as an application for Preservation funding, Rehabilitation funding, or Reconstruction funding. The descriptions below are intended to be general descriptions of the primary work being proposed. Project scope may include other secondary (lesser) work as needed. Detailed information and guidance regarding the specific project scopes listed below may be found extensively throughout the Bridge Manual.

Preservation Project Scopes may include:

Thin Polymer Overlay – A polymer resin with broadcast aggregate, applied in two separate layers. Total thickness is typically 1/4" to 3/8" thick.

Polyester Polymer Concrete Overlay – A pre-mixed polymer and aggregate concrete. Total thickness is typically 3/4" to 1" thick.

Hot Mix Asphalt Overlay with Membrane – An asphaltic concrete placed on top of a waterproof membrane. Total thickness is typically 2" to 3" thick. (The surface of this overlay may be