Geotech 7-2 presented an introduction into the typical foundations used on WisDOT structures. More detailed information is presented below. Design of the two foundation types (deep and shallow) will be discussed separately, with deep foundations further separated into three categories.

7-3.1 Shallow Foundations

Shallow foundations must provide adequate resistance against geotechnical and structural failures, and also limit deformations to within tolerable limits. Design considerations include: scour, frost action, external/surcharge loads, deformation (settlement) and angular distortion, bearing resistance, eccentricity, sliding resistance, global stability, uplift resistance and effect of groundwater, sloping ground surface, etc.

7-3.1.1 Minimum Embedment

To minimize potential frost issues, shallow foundations generally have a minimum depth of embedment of 4.0 feet.

7-3.1.2 Scour

Scour must be considered for any spread footing placed in or adjacent to a watercourse that has the potential for flowing water to cause scour. During times of flooding, volumes and velocities of the water may increase to the point where a spread footing could be undermined and caused to fail. Where the potential for such conditions exist, spread footings should be considered only if they can be founded on hard, sound bedrock and embedded into the bedrock surface at least a foot. Rip-rap protection may also be used as an added measure to reduce the potential for scour to occur, but its benefit is not considered when determining the required foundation embedment during design.

7-3.1.3 Settlement

Settlement of shallow footings needs to be computed and compared to WisDOT acceptable values. Both total, and differential, settlement are important. Current Departmental policy is to allow up to 1.5” of total estimated settlement for each substructure unit at the Service Limit State. Angular distortion also needs to be investigated as per AASHTO design requirements. The construction sequence should be considered when determining these values. Settlement should not be an issue for footings founded on competent bedrock. However, for any footing founded on natural, unconsolidated soil material above the bedrock surface, the geotechnical engineer should conduct an analysis to assure that the total and differential settlement limits are not exceeded.

7-3.1.4 Bearing Resistance

If a spread footing is to be founded on a rock surface, the geotechnical engineer must provide a value for the factored bearing resistance of the bedrock or competent material. This can be a difficult task that requires the input and guidance of an experienced geologist. While there is published information giving the compressive strengths of various bedrock types, these values are based on laboratory compression tests of small competent bedrock cores, and the suggested values are usually very high. Such tests are not a reliable indication of actual bearing capacity of in-place bedrock extending over a large area such as a footing. Conditions in the bedrock mass such as joints, voids, shear zones, and differential weathering can significantly reduce the factored bearing resistance of the bedrock. These are all items that require the examination, input, and judgment of an experienced geologist. In most cases, even the reduced factored bearing resistance of a bedrock surface will significantly exceed the requirements of a spread footing supporting a pier or abutment.

Where a spread footing is to be founded on unconsolidated soil material, a full analysis of the bearing resistance of the material must be performed, so that a factored bearing resistance can be supplied to the designer. The geotechnical engineer also needs to account for sloping ground surfaces and the location of the watertable, as appropriate.

7-3.1.5 Stability

The stability of the proposed shallow footing must also be examined. AASHTO specifications provide geotechnical engineering guidance for the necessary procedures and equations to conduct this analysis. A CDR greater than 1.0 is applied to this work. Table 1 presents the AASHTO load states that should be used in the evaluation of spread footings.

Table 1 – AASHTO Load Table

Failure Checks	Load Limits
Sliding	Strength I
Eccentricity	Strength I
Bearing Resistance	Strength II
Overall Stability (Global)	Service
Settlement	Service

The above table provides the loading conditions, however there are corresponding resistance factors that are to be used in the evaluations. These resistance factors are provided in the WisDOT Bridge Design Manual and AASHTO specifications. For example, for overall stability (global) the resistance factor is 0.65 for slopes that contain a structure element.

In summary, if a spread footing is recommended, the geotechnical engineer should provide the following information to the structural engineer:

- Type of material on which the spread footing will be founded.
- Recommended elevation(s) of the base of the footing including any variations in elevation across the extent of the footing.
- Factored bearing resistance of the bedrock or other material at the recommended footing elevation. Include any necessary discussions on removal of non-competent materials that may need to be removed, resulting in lowering the required footing base elevation below the proposed elevation.
- Potential construction problems or issues such as variability of the bedrock surface, cofferdam construction or the presence of water.

7-3.2 Piles

Driven piles have been in existence for many years, and WisDOT has a long history of successful pile design and installation.

7-3.2.1 Selection of Pile Type

The selection of pile type for a given foundation is made on the basis of subsurface conditions, applied loadings, horizontal/vertical movements, method of pile installation, bridge substructure type, cost comparisons and estimated pile lengths. The major influence in pile selection is the subsurface conditions. Subsurface conditions affecting pile selection include the presence of artesians and soft soils, cobbles and/or boulders, location of bedrock or hard bearing layer, environmental/corrosion concerns, etc. Often more than one pile type may be considered for a given project site. Over the last several years, approximately 2/3 of WisDOT driven piles have been H-piles, with the remainder being cast-in-place (CIP) pipe piles.

7-3.2.2 Static Analyses

Pile capacity for friction piles may be estimated by static analysis. The geotechnical engineer must carefully review the available boring data and separate the obtained soil information from each boring into zones based on texture and relative density. The next step is to assign appropriate soil strength parameters to each of the identified zones. It is also necessary to determine the depth to static groundwater. Once this work has been completed, static analysis computations can begin. However, the method for granular (non-cohesive) soils differs from that for cohesive soils.

7-3.2.2.1 Granular (Non-cohesive) Soils

Numerous methods of static analysis of piles in granular soils have been developed. They vary from the very simple, to somewhat complex. WisDOT has used the method developed by R. L. Nordlund (Nordlund Method) with a specific adjustment for a number of years, with good results. This specific adjustment deals with fine-medium sands, and will be discussed in the following paragraphs. To assure consistency in pile length analyses and estimated pile lengths, the Nordlund Method is the standard that must be used on all WisDOT projects unless otherwise permitted by the Department. A full discussion of this method is beyond the scope of this manual, however, an excellent presentation of the Nordlund Method and its application can be found in the Federal Highway Administration Publication, FHWA NHI-16-009.

WisDOT experience has shown that the Nordlund Method does not accurately predict resistances of piles driven in areas of deep fine to medium sands found on the outwash plains along the major rivers and in the central and northern parts of the state. Experience has shown that piles in these areas tend to drive longer than what is anticipated from the static analysis. One theory holds that a wedge of dense sand builds up at the tip of the pile which disrupts the soil strata and lessens the skin friction along the pile. In practical terms, a standard 10.75 outside diameter cast-in-place pile will develop approximately 100 tons of resistance, and then fail to show any significant increase with depth, until some change in subsurface material, such as bedrock, is encountered. WisDOT has used three approaches to deal with this situation:

1. Limit the required driving resistance of the pile to approximately 100 tons. Since the piles will not be driven to the normal resistance values, the number of piles necessary for a specific substructure unit will increase. This will then increase the foundation cost.
2. Limit the soil overburden pressure used in the static analysis to the value computed at a depth equal to 25 pile diameters. The overburden pressure used for skin friction and end bearing analyses below this depth would remain constant. The end result is a pile with increased estimated length, but more nearly matching historical lengths achieved during construction.
3. Switch to an end bearing pile if bedrock, or a very dense stratum, is within a reasonable depth. This 'reasonable depth' will vary with the specific site and conditions, but a distance of 15 feet below the estimated CIP tip elevation may be used as a general guide.

When these subsurface conditions are encountered, the geotechnical engineer should present the pros and cons of each of these options, along with a specific recommendation to the structural engineer, so that an informed decision can be made.

7-3.2.2.2 Cohesive Soils

As is the case with granular soils, a number of methods of varying complexity have been developed for friction piles driven in cohesive soils. In general, all methods attempt to model adhesion between the surrounding soil and the surface of the pile. The summation of these values over the length of the pile determines load resistance. WisDOT uses the Tomlinson method, based on work by M. J. Tomlinson (1971), and detailed in Federal Highway Administration Publication FWHA NHI-16-009 for the static analysis of friction piles in cohesive soil. Unless otherwise permitted by the Department, this is the method that must be used on all WisDOT projects.

7-3.2.3 Other Design Considerations

Many other items must be considered when designing pile-supported foundations. These are discussed below.

7-3.2.3.1 Pile Configurations

WisDOT has established spacing policies for driven piles. These have been developed to allow the use of standardized substructure designs. These also address potential issues with piles being too close, or too far apart.

The minimum pile spacing in 2.5 feet, or 2.5 pile diameters, whichever is greater. Displacement piles located in cofferdams, with estimated lengths greater than 100' have minimum pile spacing of 3.0 feet. This same requirement applies to pile-encased bridge piers and pile bents. The maximum pile spacing for bridge substructure units is 8.0 feet.

Battered piles are sometimes used for additional lateral resistance. They are often used in concert with vertical piles within the same substructure unit. Both the lateral passive resistance of the soil above the footing base, and the sliding resistance developed at the base of the footing, are generally neglected in lateral computations. Battered piles are typically 1 horizontal to 4 vertical.

7-3.2.3.2 Short Piles

WisDOT policy generally requires piles to be driven a minimum of 10 feet below the original ground line, to provide adequate resistance to lateral forces. In the design stage, shallow footings may be an alternative if the minimum pile penetration cannot be achieved. Pre-boring may be another alternative. If bedrock is shallow, and a spread footing is not an option, pre-boring 3' into the bedrock, and grouting the pile in place, is acceptable without achieving 10 feet of pile penetration.

If this minimum penetration cannot be achieved during construction, the Geotechnical Engineering Unit should be contacted to provide guidance.

7-3.2.3.3 Pile Points, Pre-boring and Seating Piles

There is sometimes confusion over the related topics of pile points, pre-boring and seating of piles. The following paragraphs provide guidance on these items.

Pile points are sometimes used to assist in pile penetration in unconsolidated materials and/or minimize potential damage to piles during driving. It has been found that points generally do not increase penetration into more-competent bedrock surfaces, but may be useful to achieve a 'bite' into highly sloping bedrock or weathered bedrock zones. Points can also be used to minimize pile damage during hard driving conditions such as when gravels or cobbles, or very dense granular materials are encountered. They also generally help keep piles aligned better during driving. Points can be welded onto all types of piles. Round CIP piles generally use conical-shaped points.

Pre-boring is generally used for three reasons. The first reason is to achieve the minimum 10-foot of pile penetration. The second reason is to achieve penetration into bedrock to increase lateral tip resistance. When this is the case, generally 3 feet of penetration into rock is required. The third reason is when displacement piles are used in new embankments. In these cases, WisDOT policy is to prebore displacement piles through new embankments that are greater than 10 feet in height. In this application, pre-boring is continued to the elevation of the base of the new fill.

There are two bid items for pre-boring, one for unconsolidated materials only, and another for rock or consolidated materials (which may also include unconsolidated materials). The use of casing may be required when pre-boring. Pre-bored holes are backfilled with sand (or other engineer-approved) material, or cement grout (when in bedrock).

Seating of piles is sometimes interpreted in different ways. This is often used when end-bearing piles are placed in pre-bored holes terminating in bedrock. In these situations, specifications call for ‘firmly seating’ the pile. Seating involves placing the pile in the pre-bored hole and using some means to ensure the tip is in firm contact with the competent bedrock surface. This does not require the use of a pile hammer, which may damage the pile tip with only a few blows. Seating ensures that there are no unconsolidated materials located below the placed pile tip, while protecting the pile from potential damage due to driving on the competent bedrock surface. If seating is required into unconsolidated soils, the use of a pile hammer is required, and the pile is driven to standard pile driving criteria.

7-3.2.3.4 Scour

Generally the structural/hydraulic designer computes the estimated scour depth based on the grain size of the streambed soils and provides this value to the geotechnical engineer. The soil frictional resistance to axial load in the scour zone is assumed to be zero and must be subtracted from the total resistance of the pile during design, to arrive at the estimated pile length. The geotechnical engineer must remember that the resistance in this scour zone must be accounted for when installing, and determining the total resistance of the pile, and in the drivability analysis of the pile.

7-3.2.3.5 Pile Loads

Piles may be designed for various types of loading conditions including axial compression, uplift, and/or lateral resistance. WisDOT Geotechnical resistance values can be found in Table 11.3-1 of the WisDOT Bridge Design Manual. In addition to this, the Department has developed Table 11.3-5 of the WisDOT Bridge Design Manual, which provides typical pile axial compression resistance values for the most common piles WisDOT uses.

7-3.2.3.6 Set Up

Piles driven in cohesive soils to the required plan length, will at times develop a capacity significantly below that required for the substructure unit. This is thought to be the result of displacement and densification of the soil adjacent to the pile caused by the driving. As this displacement and densification occurs, the soil is unable to drain and an increase in pore water pressure results. This causes a reduction in adhesion between the soil and the pile, which results in a diminished load resistance of the pile. This may account for as much as a 50% reduction in resistance. Set up is not typically included in standard WisDOT friction pile design, but is sometimes used to address construction issues.

When such situations are encountered or anticipated, utilizing what is referred to as “set-up” may be considered. Pore water pressure is at its maximum and adhesion is at its minimum immediately after driving. Allowing time for the pore water pressure to dissipate and the adhesion to increase will result in an increased pile resistance. This increase is then referred to as set-up. The increase in resistance may be quite dramatic, sometimes exceeding three times the end-of-drive (EOD) capacity. However, it cannot be assumed this will occur in this manner at any particular site, and has been found to be quite variable, even over a single bridge site. If set-up is to be used, it must be proven in construction by re-driving a representative sampling of the piles within each affected substructure unit. Normally the re-driving should occur between 24 and 72 hours after the initial driving. It is necessary to use the same hammer (warmed up) as was used in the original driving, and to place between 20 and 30 blows on the pile to ascertain penetration for application of the pile driving formula. At least 10% of the total number of piles per substructure unit, with a minimum of two piles in each unit, must be tested to assure adequate pile resistance. WisDOT has developed special provisions for this work which are available from the Geotechnical Engineering Unit.

Some caution and judgment must be used when anticipating or applying set-up. It should be considered only in cohesive soils. It cannot be assumed that it will automatically happen or that some particular amount of increase in pile resistance will take place. It is critical that re-driving must take place to ascertain that set-up has occurred and the necessary pile capacity has been developed. A set procedure must be followed to assure that the re-driving adequately measures the capacity of the piles.

7-3.2.3.6 Drag Loads

Drag loads, also called negative skin friction, can occur on piles under certain conditions. Such loads occur when a pile is driven through a layer of compressible soil and a large external load, such as an approach fill, causes consolidation in the compressible layer. As the soil consolidates and moves downward in relation to the fixed pile, movement of this soil layer and any layer above it will transfer axial downward load to the pile. This can create significant problems for both friction piles and end bearing piles. For friction piles, the portion of the pile capacity developed in the compressible layer and those above it is negated, and load from these layers is added to the pile as the soil consolidates. This can significantly reduce the capacity of the pile and lead to excessive movement of the substructure unit, or even structural failure. In end bearing piles, unanticipated load is added to the pile which could cause movement or structural failure of the pile. Drag loads acting on end-bearing and friction piles are addressed differently, as described in the WisDOT Bridge Design Manual.

The geotechnical engineer needs to anticipate when the potential for drag loads exists, and develop means and methods to compensate for this condition. The first action is to examine the soil profile developed by the site borings and determine if any layers have the potential for consolidation under the proposed embankment loads. It may be necessary to obtain samples for consolidation testing to fully make this determination. If the estimated consolidation is found to exceed ½ inch, it can be assumed that drag loads will occur. The next step is to develop a plan of action to compensate for this condition. There are three approaches that can be considered to address drag loads in friction piles:

1. Complete pre-consolidation. In this method, the approach fill must be placed (with or without a surcharge) a sufficient amount of time before pile driving occurs, to allow consolidation to take place. While this is an effective method, obtaining sufficient time to allow the substrata to consolidate sufficiently is problematic. Project timelines often preclude the use of preloading. If a large surcharge is proposed to shorten the time, stability of the surcharged fill may also be an issue. These problems must be addressed before pre-consolidation is employed.
2. Reduce pile capacity. This method reduces the typical factored load that can be placed on a pile, by the capacity of computed drag load on that pile. This drag load is computed by summing the assumed frictional pile resistances above the base of the compressible layer. These resistances are then subtracted from the pile, leaving the actual factored capacity that it can support. This could seriously reduce the available support capacity for each pile, and require a large number of additional piles to adequately support the structure. It should be noted, that while the support capacity of the pile has been reduced by the drag load, the pile must still be driven to the capacity of the applied load plus the drag load. This method has practical application in cases where relatively small reductions in capacity are anticipated. To offset a portion of this reduced capacity, bigger piles may be used to provide increased resistances. But these bigger piles will also exhibit increased down drag forces.
3. Use treatments to prevent down drag. There are a number of commercially available materials and products that can be applied to a pile surface to prevent down drag loads. The surface treatments include bitumen coatings, polymer coatings, and polymer sleeves, In addition to these, oversize steel casings may be used around each pile. All of these treatments are designed to prevent adhesion of the soil to the pile through the down drag zone. These materials can be effective in preventing down drag loads if properly applied, and carried to the base of the down drag zone. The use of coatings is usually the most cost effective approach when dealing with down drag issues. The Department has had some issues with the use of bitumen coatings, and there are concerns with loss of this material as the pile is driven to the final tip elevation. The use of bitumen is not typically considered by WisDOT.

Drag loads are addressed differently with end-bearing piles, since they can tolerate additional loads that are closer to their structural capacity. This is described in detail in the WisDOT Bridge Design Manual.

The geotechnical engineer needs to carefully evaluate the subsurface site conditions, structure/pile design, time constraints, effectiveness of treatment, and cost, to recommend and/or develop the most suitable method to address down drag on piles. Close cooperation with the structural engineer is necessary to implement the selected method.

7-3.2.3.8 Corrosion

Piling should be designed with sufficient corrosion resistance to assure a minimum design life of 75 years. 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. But corrosive sites may include 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.

If corrosive sites are identified, special considerations (thicker pile shells, heavier pile sections, painting or concrete encasement) may be considered. All exposed piling should be painted.

7-3.2.3.9 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. A drivability analysis needs to be conducted on the recommended piles to ensure they can be driven to the estimated length (based on static analysis), and required driving resistance without the development of excessive driving stresses. Drivability is treated as a strength limit state.

The geotechnical engineer should perform the drivability evaluation in the design phase, based on a preliminary dynamic analysis using wave equation techniques. This analysis needs to ensure that the assumed pile driving hammers are capable of mobilizing the required nominal (ultimate) resistance of the pile at driving stress levels below the factored driving resistance of the pile. Drivability can be the controlling strength limit state check for pile design. Because of the high strain rate and temporary nature of loading conditions during driving, higher stress levels are allowed than during service conditions.

This analyses requires the designer to select an assumed pile hammer/system that the contractor may use, and appropriate soil parameters. A list of a typical hammers is provided in the WisDOT Bridge Design Manual. Any previous pile driving information and/or pile driving experience can be used to refine the current drivability analysis. The drivability analysis should account for the total resistance in all soil layers, including those that may be negated by down drag or scour effects. In addition, WisDOT reduces the soil resistance parameters by appropriate values to account for loss of soil strength during driving, as shown in Table 11.3.4 of the WisDOT Bridge Design Manual. The Department generally limits driving stresses to 90% of the steel yield strength. Results are evaluated based on an allowable hammer blow count of 25-120 blows per foot.

The results of this investigation is not intended to evaluate the ultimate pile capacity or to establish plan pile lengths. If excessive driving stresses are found, pile sizes, wall thicknesses, or the selected pile hammer can be altered.

7-3.2.3.10 Construction Monitoring

The goal of pile foundation design is to provide the most efficient and economical design for the subsurface conditions. This design is influenced by the resistance factor, which is a function of the pile resistance determination method during installation. WisDOT generally bases this factor on the use of the FHWA-modified Gates dynamic pile driving formula, but sometimes uses the driving criteria established by dynamic testing (Pile Driving Analyzer [PDA]) with signal matching (CAPWAP), or more rarely, PDA monitoring with signal matching with static load testing. Each of these methods has a different resistance factor.

On projects with larger amounts of piling quantities, the geotechnical engineer should compare the modified-Gates method with the PDA/CAPWAP method to determine which is more economical for the project. To make this comparison, the number of PDA-tested piles can be reduced using the same Required Driving Resistance (RDR) values as the modified-Gates method, but allowing higher Factored Axial Compression Resistance (FACR) values due to the higher PDA resistance value. This cost comparison needs to include pile quantities, PDA monitoring/testing/analyses costs, and the contractor's cost for delaying operations due to testing, set-off time, redriving, and analyses time of the test piles. Wisconsin bridge contractors do not currently have the equipment to conduct PDA testing/analyses, so must subcontract this work when it is included in a let project.

7-3.3 Drilled Shafts

Design methodologies for drilled shafts can be found in LRFD 10.8 of the "Drilled Shafts and Drilled Shafts: Construction Procedures and LRFD Design Methods", FHWA Publication NHI-10-016, FHWA GEC 010, 2010. Strength limit states for drilled shafts are evaluated the same way as for driven piles. Evaluation of axial resistance, combined axial and flexure, shear and buckling are required. Resistance factors for drilled shafts are selected based on the method used to determine the nominal (ultimate) resistance capacity of the drilled shaft. The geotechnical resistance factor is based on the intended method of resistance verification in the field. For pier columns containing at least 5 shafts, the AASHTO LRFD resistance factors can be increased by 20%. For piers supported on a single shaft, the AASHTO LRFD resistance factors must be decreased by 20%.

Most drilled shafts provide geotechnical resistance in both end bearing and side friction. Since 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. Consideration of deflection (service limit state) may control over the axial geotechnical resistance, since displacements required to mobilize the ultimate end bearing can be excessive. More detailed design guidance is provided in the WisDOT Bridge Design Manual.

7-3.4 Micropiles and Augercast Piles

Constructability issues often dictate when micropiles or augercast piles are necessary. Design of both of these types of piles is beyond the scope of this manual and can be found in the AASHTO LRFD Design Manual, WisDOT Bridge Design Manual, FHWA Design Circulars, and FHWA Publication No. FHWA-NHI-05-039.