SECTION 10
FOUNDATIONS

10.1 GENERAL SCOPE

Design of structure foundations shall be in accordance with AASHTO, project contract documents, and CDOT Geotechnical Design Manual, unless otherwise specified in this Section of the BDM.

10.2 GEOTECHNICAL INVESTIGATIONS

Geotechnical investigations shall be conducted in accordance with AASHTO and the guidance provided in the CDOT Geotechnical Design Manual.

10.2.1 Ring-Lined Split Barrel Sampler

The 2.5-in. outside diameter, ring-lined split barrel sampler, often referred to as the Modified California (MC) sampler, is routinely used in Colorado to obtain disturbed samples of cohesive soil/rock for swell testing.

If penetration resistance values (blow counts) obtained using an MC sampler are used in conjunction with correlations based on standard penetration test (SPT) resistance values (N-values), the penetration resistance values should be corrected to account for the size of the MC sampler (see Fang, 1991), as appropriate based on the judgment of the Geotechnical Engineer.

In general, it is preferable to use SPT resistance values in SPT-based correlations rather than to correct MC penetration resistance values.

10.2.2 Energy Measurements for Sampling Hammers

The energy delivered to drill rods when conducting SPT and MC sampling can vary significantly depending on factors, including the type of sampling hammer, the general condition of the hammer, and the operator. Therefore, CDOT requires the use of sampling hammers that have been tested to determine the actual energy transfer to the drill rods.

All sampling hammers used to complete field explorations for CDOT projects shall be tested to determine the energy transfer ratio (the measured energy transferred to the drill rods divided by the theoretical potential energy of the sampling hammer) in accordance with ASTM D4633. The testing shall be completed no more than two years before the date of sampling.

The project geotechnical report or the individual boring logs shall indicate the energy transfer ratio. The energy transfer ratio shall also be reported on the geology sheet. In addition, the geology sheet shall indicate whether the reported penetration resistance values are raw values or values that have been corrected for hammer efficiency.

As appropriate for use in geotechnical evaluations, the Geotechnical Engineer should correct penetration resistance values to an equivalent hammer efficiency of 60 percent (N_{60} values).

10.3 LIMIT STATES AND RESISTANCE FACTORS

10.3.1 Service Limit State

Foundation design at the service limit state shall be in accordance with AASHTO.

10.3.2 Strength Limit State

Resistance factors at the strength limit state for foundation design shall be in accordance with AASHTO, unless otherwise indicated in this Section of the BDM.

10.3.3 Extreme Event Limit State

As specified by AASHTO, resistance factors for the extreme event limit state, including earthquake, ice, vehicle, or vessel impact loads, shall be taken as 1.00. For uplift resistance of piles and shafts at the extreme event limit state, the resistance factor shall be taken as 0.80 or less.

AASHTO 10.5.5.3.3

10.4 SPREAD FOOTINGS

10.4.1 General

The Designer shall evaluate the suitability and applicability of spread footing foundations on a case-by-case basis.

10.4.2 Footing Embedment

The base of spread footings on soil shall be embedded below the local or regional frost depth, with a minimum embedment of 3 ft. The minimum embedment of spread footings on bedrock may be reduced to less than 3 ft. based on the recommendation of the Geotechnical Engineer.

AASHTO 10.6.1.2

For establishing spread footing embedment into stream banks based on scour considerations, see Section 2.11.2 of this BDM.

The requirements of this section do not apply to MSE wall footers. Refer to current Staff Bridge Worksheets for MSE Walls for MSE wall requirements.

10.4.3 Tolerable Movements

Tolerable foundation movements shall be in accordance with AASHTO. As noted by AASHTO, angular distortions between adjacent foundations should not exceed 0.008 radians in simple spans and 0.004 radians in continuous spans.

AASHTO 10.5.2.2, C10.5.2.2

Consistent with AASHTO, transient loads may be omitted from time-dependent settlement analyses at the Service I Load Combination.

10.5 DRIVEN PILES

10.5.1 General

10.5.1.1 Pile Types

Driven H-piles are frequently used to support structures in Colorado. In most applications, H-piles are driven to practical refusal on bedrock. H-pile sections are supplied standard as Grade 50 steel (f_y = 50 ksi).

For bridges, the most readily available H-pile sections include:

- HP 14x89
- HP 12x74
- HP 12x53

Other H-pile sizes may be used when availability is verified with local suppliers and when any delays due to custom pile orders do not negatively affect the project schedule.

Although less frequently used in Colorado, other pile types may be feasible and preferable to H-piles depending on project requirements. For instance, closed-end pipe piles may be advantageous at sites with relatively deep bedrock, where a closed-end pipe pile may develop greater axial resistance at shallower depths compared to a comparable H-pile section. Sheet piles may be used for foundation support, especially for projects where such use may benefit the construction schedule or cost.

When using a less common pile type, the Designer shall confirm that the selected pile section is available from local suppliers.

10.5.1.2 Battered Piles

AASHTO 10.7.1.4

Battered piles may be used to increase lateral resistance of driven piles. The Designer should consider that battered piles will provide a stiffer lateral response than that of vertical piles.

Where used, the preferred pile batter is 1 horizontal to 6 vertical (1H:6V). The maximum batter of driven piles shall not exceed 1H:4V due to constructability considerations.

Piles less than 15 ft. in length and driven to refusal on bedrock shall not be battered.

10.5.1.3 Embedment

The Designer should consider the potential for piles to encounter refusal on bedrock or obstructions, such as boulders, before reaching the depth required for stability under axial and lateral loading. The Designer may specify a minimum tip elevation on the plans to address this issue. Pre-boring may be used in cases where refusal is anticipated to occur above the required minimum tip elevation, although the Designer should consider using other foundation types that may be preferable in terms of design or constructability.

10.5.1.4 Corrosion of Piles in Soil/Rock

In aggressive soil/rock, the Designer shall incorporate appropriate corrosion mitigation measures. Acceptable corrosion mitigation measures for driven piles include the use of sacrificial steel, concrete encasement, and factory-applied coatings in combination with a reduced thickness of sacrificial steel. Field-applied coatings shall not be used, except as repairs to factory-applied coatings. Weathering steel is not considered a mitigation measure for corrosion. **AASHTO 10.7.5**

In general, corrosion of steel piles is greatest in soils that have been disturbed, that is, where earthwork activities have occurred. Compared to undisturbed soils, disturbed soils have increased oxygen content, which supports corrosion. In undisturbed soils, corrosion may occur in the zone of unsaturated soil above the groundwater table. Corrosion may be exacerbated in the zone of fluctuation of the groundwater table. Significant corrosion does not generally occur in undisturbed soil/rock below the groundwater table.

In soil/rock above the groundwater table, the Geotechnical Engineer shall conduct corrosion testing of representative soil/rock samples. If any of the following conditions exist, the soil/rock shall be classified as aggressive:

- Resistivity is less than 2,000 ohm-cm.
- pH is less than 5.5.
- pH is between 5.5 and 8.5 in soils with high organic content.
- Sulfate concentration is greater than 1,000 parts per million (ppm).
- Chloride concentration is greater than 500 ppm.

Where corrosion testing indicates aggressive soil/rock, the Geotechnical Engineer shall indicate the elevation range(s) where the aggressive soil/rock is anticipated based on test results.

Where aggressive soil/rock is present, the thickness of sacrificial steel shall be calculated based on a minimum corrosion rate of 0.001 in. per year. Published corrosion rates vary widely. The specified minimum corrosion rate is based on criteria established by the California Department of Transportation (2013), the US Army Corps of Engineers (2012), and the Florida Department of Transportation (2016).

The Designer shall assume that corrosion occurs over all steel surfaces in contact with the aggressive soil/rock. Corrosion rates greater than the minimum value specified herein may be appropriate, particularly where piles are installed in landfill materials, cinder fills, organic soils, or mine waste/drainage. Corrosion mitigation is not required in soil/rock below the groundwater table.

If factory-applied coal-tar epoxy coating is used for corrosion mitigation, the coating shall be assumed to be effective for 30 years. In calculating the sacrificial steel thickness, the Designer shall assume corrosion begins after the first 30 years and continues through the remaining design life, as appropriate. If protective coatings are used, the Geotechnical Engineer shall provide appropriate axial design parameters accounting for a potential reduction in side resistance.

Sacrificial steel is not necessary where concrete encasement is used for corrosion mitigation. Piles protected by concrete encasement should be coated with a dielectric coating near the base of the concrete jacket.

AASHTO C10.7.5

10.5.1.5 Corrosion of Piles Exposed to Atmospheric Conditions

The following provisions apply only to situations where piles are extended above the ground, such as sheet pile abutments or H-pile/pipe pile piers.

For non-weathering steel piles, aggressive conditions shall be assumed for the first 5 ft. of pile below grade and for the entire portion of the pile exposed to atmospheric conditions.

Corrosion mitigation is not required for weathering steel piles exposed to atmospheric conditions and not located within the splash zone or underneath a bridge expansion joint.

Corrosion mitigation for the remaining portion of piles embedded in soil/rock shall be as required in Section 10.5.1.4, for both non-weathering and weathering steel piles.

10.5.1.6 Pile Cap Embedment

For establishing pile cap footing embedment into stream banks based on scour considerations, see Section 2.11.2 of this BDM.

10.5.2 Geotechnical Design and Analysis

AASHTO 2.6.4.4.2

10.5.2.1 Point Bearing Piles on Rock

Piles that will penetrate the bedrock 3 ft. or more shall be designed in accordance with the requirements specified by AASHTO for “Piles Driven to Soft Rock.” Piles that will penetrate the bedrock less than 3 ft. shall be designed in accordance with the requirements specified by AASHTO for “Piles Driven to Hard Rock.”

AASHTO 10.7.3.2.2 and 10.7.3.2.3

In general, it is anticipated that piles driven into the relatively weak sedimentary bedrock typically encountered along the Front Range would classify as “Soft Rock,” while piles driven to higher strength bedrock where significant bedrock penetration is not typically achieved would classify as “Hard Rock.”

Pile protection (tips, points, or shoes) shall be included for all piles driven to bedrock.

10.5.2.2 Small Groups of Piles

At the strength limit state, the resistance factor for geotechnical axial resistance shall be reduced by 20 percent for groups of piles containing three or fewer piles, unless otherwise approved by Unit Leader in coordination with the Foundation SMEs.

AASHTO C10.5.5.2.3

10.5.2.3 Drivability Analysis

CDOT Standard Specification 502 provides requirements for pile drivability analyses (wave equation analysis of pile driving [WEAP]). The Contractor typically completes WEAP.

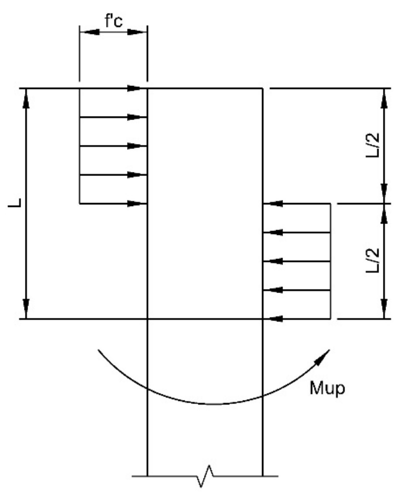
AASHTO 10.7.8

The Geotechnical Engineer should consider completing WEAP during the design phase when:

- A pile type, section, or driving procedure not routinely used in local practice (see Section 10.5.1.1) is proposed.
- A pile with an axial resistance greater than what is typically used in local practice or which may require the use of a pile driving hammer larger than typically used in Colorado (nominal resistance greater than approximately 500 kip) is proposed.
- A pile will be driven into a relatively deep bearing layer such that the driving resistance is likely to exceed the required geotechnical axial resistance (over-driving).

10.5.3 Top of Pile Fixity

The following simplified method may be used to calculate the minimum pile embedment required to classify the connection at the top of the pile as fixed.



$$M_{up} = \varphi f'_c b_f \left(\frac{L}{2} * \frac{3L}{4} - \frac{L}{2} * \frac{L}{4}\right)$$

$$M_{up} = \varphi f'_c b_f L^2 \left(\frac{3}{8} - \frac{1}{8}\right)$$

$$4M_{up} = \varphi f'_c b_f L^2$$

$$L = \sqrt{\frac{4M_{up}}{\varphi f'_c b_f}}$$

Figure 10-1: Pile Fixity

Where:

L = Required pile embedment into cap (in.)

ϕ = Strength reduction factor for concrete bearing = 0.7 (AASHTO 5.5.4.2)

f’c = 28-day compressive strength of concrete (ksi)

M_{up} = Plastic moment capacity of pile about strong axis (kip-in.)

b_f = Pile flange width (in.)

Table 10-1 presents the calculated embedments for the most common HP shapes, based on a ϕ of 0.7 and f’c of 4.5 ksi.

Table 10-1: Calculated Embedments

HP Pile Section	Minimum Embedment (in.)
12x53	20
12x74	24
14x89	26

For specific criteria regarding pile embedment at integral abutments, see BDM Section 11.

10.5.4 Field Splice

The Designer shall note on the plans the elevation above which complete joint penetration (CJP) welds are required for the flanges of all H-pile field splices. The Designer shall also note on the plans that below this elevation, partial joint penetration (PJP) flange welds or other commercially available splices using mechanical connections may be permitted upon review by the Engineer. The elevation shall be taken as the lowest primary moment inflection point in the pile obtained from all load combinations producing bending moment in the pile, including scour and extreme event load cases (see Figure 10-2). At the Designer’s discretion, piles that are not subjected to significant bending moment may be exempt from this provision.

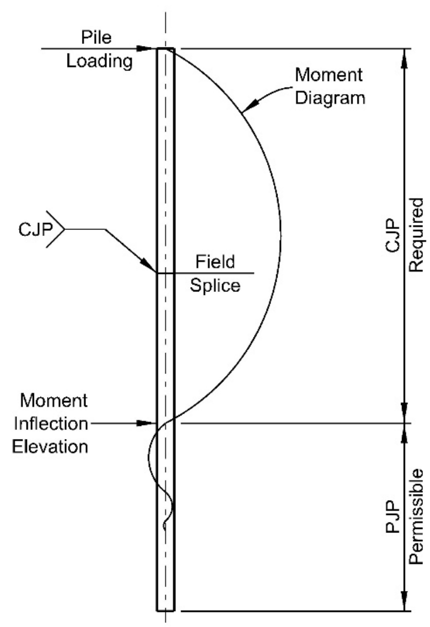


Figure 10-2: Moment Inflection Point and H-Pile Field Splices

10.5.5 Dynamic Testing

As required by AASHTO and CDOT Standard Specification 502, dynamic testing shall be completed during pile installation to monitor potential pile damage, to determine axial resistance, and to establish driving criteria.

AASHTO 10.7.3.8.3

In accordance with AASHTO, higher resistance factors for geotechnical axial resistance may be used if dynamic testing is completed during pile installation. The Designer should note that for bridges with more than 100 piles, the test frequency required by AASHTO to use a resistance factor of 0.65 is more stringent than the test frequency required by CDOT Standard Specification 502. Therefore, if a resistance factor of 0.65 is used for a bridge with more than 100 piles, a Project Special Provision is required to modify the dynamic testing frequency indicated in the Standard Specification to maintain compliance with AASHTO.

AASHTO Table 10.5.5.2.3-1

10.5.6 Load Testing

Load testing (axial or lateral) may be conducted to justify the use of increased resistance factors and to reduce uncertainty in the performance of driven piles. During the structure selection process, the Designer shall review and evaluate the need, benefits, and feasibility of conducting load testing.

When load testing is completed, the entity completing the load test shall prepare a report sealed by a professional engineer licensed in the State of Colorado summarizing test results.

10.6 DRILLED SHAFTS

The term “drilled shaft” as used herein is interchangeable with drilled pier, drilled caisson, and other similar terms.

10.6.1 General

10.6.1.1 Geometry and Dimensions

Drilled shafts used to support bridges and retaining walls shall have a minimum diameter of 24 in. Drilled shafts used to support sound walls shall have a minimum diameter of 18 in. Length to diameter ratios, L/D, are typically less than 25.

Where a drilled shaft supports a single column, the top of shaft shall be embedded a minimum of 2 ft. below ground surface, unless the Geotechnical Engineer recommends deeper embedment.

In contrast to AASHTO, CDOT allows the use of drilled shafts that are smaller in diameter than the columns they support. This allows constructability advantages, such as eliminating the need for separate column dowels embedded into the caisson.

AASHTO 10.8.1.3

10.6.1.2 Tip Elevation

The Designer shall add a note on the plans requiring drilled shafts to be advanced to the estimated tip elevation or to the minimum penetration into bedrock, whichever produces the lower tip elevation. No allowance will be made to terminate the drilled shafts above the estimated tip elevation on account of encountering bedrock above the anticipated elevation or any other circumstances.

10.6.2 Geotechnical Design and Analysis

10.6.2.1 Axial Resistance in Weak Rock

Rock-socketed drilled shafts are frequently used in Colorado. SPT-based methods are often used to estimate the axial resistance of sedimentary bedrock encountered along the Front Range. For sites with bedrock N-values typically between 20 and 100 blows per foot, the “soil-like claystone” design procedure described by Abu-Hejleh et al. (2003) may be used to determine nominal unit side resistance and end bearing values.

The resistance factor of 0.75 recommended by Abu-Hejleh et al. (2003) for the “soil-like claystone” method shall not be used because this value exceeds typical resistance factors specified by AASHTO, including the maximum resistance factor of 0.70, which assumes load testing is completed.

AASHTO Table 10.5.5.2.4-1

A resistance factor of 0.60 shall be used with the “soil-like claystone” method (Abu-Hejleh et al., 2003). The resistance factor was calculated by fitting to allowable stress design (ASD) assuming the following:

- Ratio between permanent and live loads of 3:1
- Permanent Load Factor of 1.25
- Live Load Factor of 1.75
- Factor of Safety of 2.25

For sites with bedrock N-values typically greater than 100 and where rock coring produces suitable core recovery (i.e., samples can be recovered for strength testing and the rock mass can be characterized to an appropriate degree), it is preferable to evaluate axial resistance using design methods based on the unconfined compressive strength, as described in AASHTO and FHWA Report No. FHWA-NHI-10-016 (Brown et al., 2010).

AASHTO 10.8.3.5

10.6.2.2 Roughening and Shear Rings

Roughening may be completed to remove smeared or disturbed materials from the sides of drilled shaft excavations. The Geotechnical Engineer shall indicate when roughening is required. Roughening is difficult to inspect and should be used only when approved by Unit Leader in coordination with the Foundations SMEs.

Because shear rings are difficult to inspect, they shall not be used unless approved by Unit Leader in coordination with the Foundations SMEs.. As an alternative to using shear rings to increase axial resistance, the drilled shaft could be lengthened or increased in diameter.

10.6.3 Non-destructive Integrity Testing

10.6.3.1 Test Methods

Cross-hole sonic logging (CSL) is an acceptable non-destructive method to evaluate the integrity of completed drilled shafts. Thermal Integrity Profiling (TIP) may be used with approval from Unit Leader in coordination with the

Foundations SMEs. If TIP is specified, the designer shall prepare an appropriate Project Special Provision.

Methods based on the analysis of stress waves, such as sonic echo and impulse response, shall not be used as the primary test method unless access tubes are unavailable.

All testing shall be completed in accordance with the applicable ASTM standards.

10.6.3.2 Test Frequency

The requirements presented in this section are only applicable to drilled shafts used as bridge foundations. The frequency of integrity testing for drilled shafts used in other applications (retaining structures, landslide stabilization, etc.) shall be at the discretion of the Designer, as approved by Unit Leader in coordination with the Foundations SMEs. As necessary for non-bridge applications, the Designer should prepare a Project Special Provision to specify the desired test frequency.

CSL access tubes shall be installed in all non-redundant drilled shafts. With respect to CSL testing requirements, a non-redundant drilled shaft is defined as any drilled shaft at an abutment or a pier supported by two or fewer drilled shafts. CSL access tubes shall also be installed in all drilled shafts to be constructed in a water crossing and in all drilled shafts that will be constructed in soil/rock requiring the use of temporary excavation support (i.e. casing or drilling fluid). At the discretion of the Designer, other drilled shafts on the project may be selected to require CSL testing, such as largely spaced shafts.

CSL testing shall be completed on all non-redundant drilled shafts. CSL testing shall be completed on a minimum of 50 percent of drilled shafts equipped with CSL access tubes. Testing locations shall be at the discretion of the Engineer. If CSL testing indicates anomalies, the remaining drilled shafts at the pier/abutment shall also be tested.

Installation of CSL access tubes and integrity testing are not required for drilled shafts with permanent casing socketed into bedrock, regardless of redundancy or shaft location.

Other agencies, such as railroads, may have more stringent testing requirements. The designer shall determine if any non-CDOT entities have applicable testing requirements.

The Designer shall indicate in the plans the minimum number of drilled shafts to be tested.

10.6.3.3 Addressing Anomalies

Anomalies indicated by CSL testing shall be addressed in accordance with Standard Specification 503.

Guidance on repairing drilled shaft anomalies is described in FHWA Report No. FHWA-NHI-10-016 (Brown et al., 2010). Additional information is provided in the ADSC – IAFD Standard Drilled Shaft Anomaly Mitigation Plan

(Association of Drilled Shaft Contractors – International Association of Foundation Drilling, 2014).

If test methods other than CSL are proposed, the Designer shall specify criteria for the evaluation and acceptance of test results in a Project Special Provision.

10.6.4 Load Testing

Load testing (axial or lateral) may be conducted to justify the use of increased resistance factors and to reduce uncertainty in the performance of drilled shafts. During the structure selection process, the Designer shall review and evaluate the need, benefits, and feasibility of conducting load testing.

When load testing is completed, the entity completing the load test shall prepare a report sealed by a professional engineer licensed in the State of Colorado summarizing test results. The report shall include all necessary information and data to enter the test into the DSHAFT load test database (see Garder et al., 2012).

10.7 REFERENCES

CDOT Research, 2003, Improvement of the Geotechnical Axial Design Methodology for Colorado's Drilled Shafts Socketed in Weak Rocks, Report No. CDOT-DTD-R-2003-6.

Federal Highway Administration, September 2018, Drilled Shafts: Construction Procedures and Design Methods, Publication No. FHWA-NHI 18-024.

Colorado Department of Transportation, 2021, Geotechnical Design Manual.