Chapter 4
Substructures

Introduction

This chapter provides guidance on the design of abutments and intermediate substructures.

Abutment Types

INTEGRAL ABUTMENTS consist of a concrete cap placed on a single row of flexible steel piles. The girders are encased into a concrete diaphragm that is integral with the concrete cap. Integral abutments are the preferred abutment type because they eliminate expansion joints on the bridge. Integral abutments shall be limited to bridge lengths of 300’ or less and skews less than or equal to 30 degrees unless approved by Bridge Program Staff.

SEMI-INTEGRAL ABUTMENTS consist of a concrete cap on a foundation. The girders are encased in a concrete diaphragm that is supported by expansion bearings placed between the cap and the diaphragm. Semi-integral abutments can be considered for bridges where integral abutments are not feasible.

SILL TYPE ABUTMENTS consist of a concrete footing with a backwall and girder pedestal(s) supported by the footing. Bearings at sill type abutments can be fixed bearings or expansion bearings. An expansion joint is required between the backwall and the end of the slab when expansion bearings are used. An expansion joint is not required for fixed bearings. Lateral support of girders is provided by cross frames between girders. This type of abutment requires an expansion joint and should only be used where integral or semi-integral abutments cannot be used.

Intermediate Substructure Types

SOLID SHAFT PIERS , often times referred as hammerhead piers, consist of a single solid concrete cross section that supports a cap. A pier wall is a common variation that consists of a solid concrete cross section that directly supports the girder bearings.

The major axis of the pier is parallel to the feature intersected. A circular or smaller rectangular cross section may be used when the stream flow is not in the same approximate direction as the major axis. Solid shaft piers are usually supported by a spread footing, or a footing supported on drilled shafts or piles, but occasionally can be supported by a single row of drilled shafts or piles.

Solid shaft piers should be considered where there are large lateral forces anticipated (ex. Stream, Ice, or Collision) or when the slenderness of individual columns on a multi-column bent becomes a concern.

MULTICOLUMN BENTS, occasionally referred to as frame bents, have two or more concrete columns that support the cap. Consideration should be given to using multi-column bents on wider structures. Since water is allowed to flow between the columns, care must be used to ensure that debris collection is not a problem.

Multicolumn bents can be supported by spread footings, footings on drilled shafts, footings on piles, or drilled shafts. Multicolumn bents can be supported on a combined footing when the column spacing is close, or on isolated footings when the column spacing is greater.

Multi-column bents are the most common type of substructure and are typically more cost effective than a solid shaft pier.

PILE BENTS are a variation of multicolumn bents, except that they are made of pile (HP) sections rather than concrete columns. The cap usually consists of cast in place concrete. Consideration should be given to using pile bents on low height, short span structures. This substructure type should not be considered where significant lateral forces are anticipated

Abutment Design

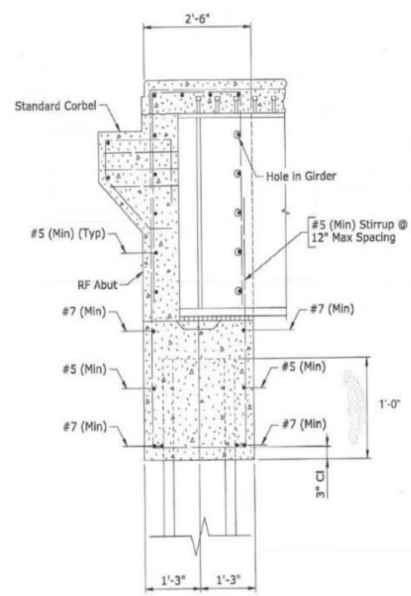
GENERAL
Abutment designs shall be investigated for any combination of forces which may produce the most severe condition in accordance with the load combinations. Where abutments support the approach slabs, one third of the approach slabs self weight shall be included in the abutment self weight Wind loading on abutments is negligible and shall be ignored. For simple span bridges with integral abutments, seismic forces can typically be ignored except the stirrups connecting the abutment cap to the diaghargm shall be adequate to resist the anticpated lateral forces.

The riprap countermeasures designed by the Hydraulics Section should be assumed to be adequate for providing slope protection for all flood events. Therefore, the steel piling or drilled shafts supporting abutments may be assumed to have full soil support for their entire length as long as countermeasures are utilized.

INTEGRAL ABUTMENTS Integral abutments resist thermal expansion by hinging of the steel piles directly below the cap. For shorter to medium length bridges, the size of piling will typically be controlled by the axial capacity of the pile (either structural or soil) Reinforced backfill consisting of reinforced layers of special granular backfill shall be placed behind integral abutments. The gap between the reinforced fill and abutment cap allow the abutment to expand without increasing the soil pressure. For bridges without approach slabs, for example gravel roads, reinforced backfill may be omitted. Where reinforced backfill is omitted, special consideration shall be given to the earth pressure applied to the wingwalls.

See the diagram below for standard configuration of integral abutments. The minimum dimensions shown shall be used unless special circumstances require modification. When feasible, piles shall be placed directly underneath each girder. The minimum length of piling below the abutment cap shall be 15'-0". Pre-drilling holes will be required if pile refusal is anticipated within this length.

The required reinforcing steel will be determined in the design, however, the reinforcing steel shown in the diagram shall be considered a minimum. Number 4 stirrup bars at 6" minimum spacing may be used substituted for the number 5 stirrup bars for shallow girders to achieve the required lap length of the cap to diaphragm stirrup bars.



SEMI-INTEGRAL ABUTMENTS

Semi-integral abutments shall have caps similar in geometry to integral type abutments. When feasible, piles or drilled shafts shall be placed directly underneath each girder. Shear keys are required to restrain the girders. Reinforced fill is required behind the diaphragm to allow the superstructure to expand.

SILL TYPE ABUTMENTS

Sill type abutments shall resist overturning about the toe of the footing, sliding, or overloading of piles at the point of maximum pressure. Full lateral earth pressure shall be applied to sill type abutments even if reinforced backfill is utilized. Passive earth pressure that would help resist the earth pressure shall be ignored.

Sill type abutments shall also be evaluated for construction load cases. Superstructure loading that would help resist earth pressure shall be ignored in this case.

Where steel rocker bearings are used, sill type abutments shall be evaluated for friction forces from the bearings

Wingwalls

There are two types of wingwalls used in abutments, elephant ear wingwalls and sweptback wingwalls. Reference Section 4.07 – Abutments of the Bridge Applications Manual for geometry and guidance on determining the length of the wingwalls. Elephant ear wingwalls are preferred over swetpback wingwalls. Sweptback wingwalls should be considered when fill slopes wrapping around elephant ear wingwalls will spill over the front edge of berms. Where reinforced backfill is used, the design of the wingwalls shall be based on an active earth pressure.

Intermediate Substructure Design

GENERAL

Intermediate substructures designs shall be investigated for any combination of forces which may produce the most severe condition in accordance with the load combinations. For bridges in seismic category A, the horizontal force applied to the anchor bolts shall not be applied to the substructure.

The following shall be used to design foundations at intermediate substructure locations for scour: The Extreme Event Limit state shall be used for the check scour (typically 500 year event) the Strength Limit State shall be used for the design scour (typically 100 year event). The design flood scour elevation (typically 100 year event) shall be shown on the General Plan and Elevation sheet.

WYDOT Hydraulics Program uses a conservative approach for determining the design and check flood scour flood frequencies, and in most instances, the check scour elevation is within a few feet of the design scour elevation. In the case where the check flood scour elevation is within 5 feet or less of the design scour elevation, the foundation shall be designed using the design scour elevation (typically the 100 year event) for strength limit state, and the substructure does not need to be evaluated for the check flood scour for the extreme event limit state. If applicable, ice shall be checked for the extreme event limit state utilizing ½ of the estimated scour depth determined from the design scour (typically 100 year event).

Pile bents will be the exception since pile buckling is very sensitive to this depth. In most cases, however, pile bents will not be utilized on bridges with large scour depths.

Where the check scour is 5 feet or more in depth from the design scour elevation, the above strategy will be discussed with the Bridge Program Staff on a case by case basis and documented in the Structure Selection Report.

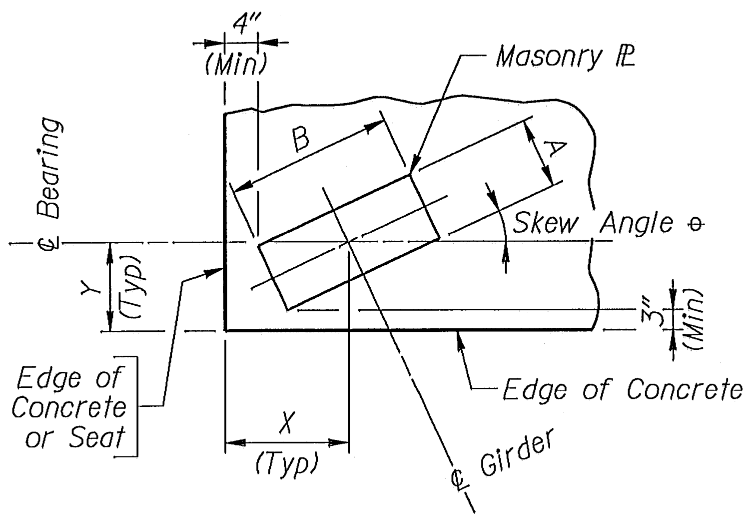
Bent caps, columns, and footings have historically been designed using the sectional method rather than refined analysis such as finite element modeling or strut and tie modeling. Historical inspection data within the state suggests that the previous practices have resulted in adequate designs. The Bridge Applications Manual provides good guidance on reinforcing steel design including development of bars at nodes. Therefore, sectional method analysis in combination with standard practices outlined in the Bridge Applications Manual will be adequate for most substructure components. The engineer is responsible for identifying structural members that may require a more refined analysis such as deep beams, straddle bents, and corbels etc. The provisions for Article 5.8.2.6 shall be required for sectional method analysis.

In multi-column bents, if the stiffness of the column is 15% or less than the stiffness of the cap, the moment transfer from the bent cap to the columns may be considered negligible and the cap may be designed as a continuously supported beam. When considering slenderness effects for this approximate method, the ratio of maximum factored axial permanent load to maximum factored total axial load shall be used for βd calculations in lieu of factored moment.

The cap width shall be of sufficient dimension to minimize the chance of concrete spalling when the loads are placed. The figure below shall be used as the basis for calculating bridge cap geometry. The designed dimensions of the cap and bearing seats should be rounded up to the nearest 6". Cap widths shall be a minimum of 3" greater than the column diameter.

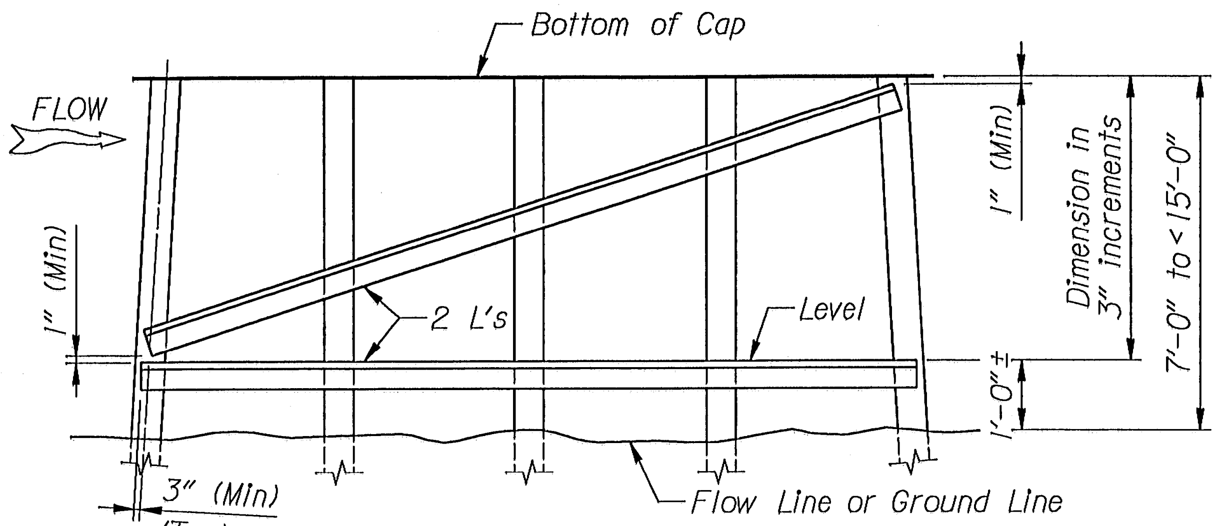
$$X = 4'' + \frac{A}{2}(\sin\theta) + \frac{B}{2}(\cos\theta)$$

$$Y = 3'' + \frac{A}{2}(\cos\theta) + \frac{B}{2}(\sin\theta)$$

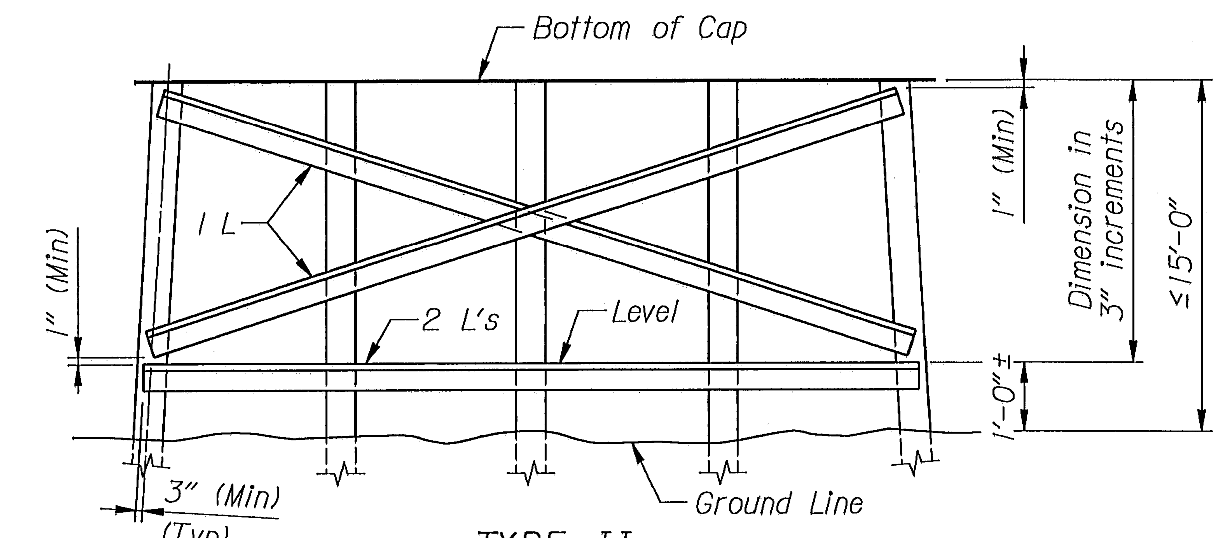


Anchor Bolts shall be used to securely anchor the superstructure to the substructure. For more information on anchor bolt sizes and required holes, see Section 4.09 - Superstructure in the Bridge Applications Manual. The clearance from the edge of the concrete seat to the edge of an anchor bolt should be 6" minimum. At least one shear stirrup is to be placed between the anchor bolt and the end of seat. The top mat of reinforcing steel shall have a 90 degree hook at the ends of the cap.

Pile support angles are required on pile bents when the distance from the flow line or ground line to the cap bottom equal or exceeds 7'-0". Type I cross frames are required when the distance is greater than or equal to 7'-0" but less than 15'-0". A Type II cross frame is required when the distance is greater than or equal to 15'-0", but is generally not used in streams and rivers. If possible, the length of the support angles should not exceed 40'-0".



TYPE I



TYPE II

Spread Footing Design

SPREAD FOOTINGS shall be designed to keep the maximum soil pressures within tolerable nominal soil resistance for both bearing strength and settlement. Spread footings shall also be proportioned to maintain stability for all limit states including overturning, sliding, and uplift. The footing shall be proportioned to eliminate uplift under the Service Limit State.

The bottom of footing depths shall be determined with respect to the character of the foundation materials and the possibility of scour. Spread footings shall be founded at a depth determined by scour analysis and below frost. At a minimum, the bottom of the footing shall be placed 5'-0" below ground. A minimum ground cover of 1'-0" above the top of the footing is required for all footings. Footings placed on steel piling or drilled shaft foundations shall be considered where there is potential for scour below footing depth, poor soil bearing resistance, or non-scourable rock is not close to the ground surface.

The Geology Report will provide factored bearing charts. The values are based on an assumed footing size and should be considered preliminary. Service limit state will typically control the plan size of the footing. The Geology Program will provide final values once a preliminary footing size has been determined. In most cases this should only require one iteration.

If possible, the top of spread footings on piles or drilled shafts shall be placed below scour elevation. If this is not practical, the Hydraulics Program shall be notified to determine the effects that the footing has on pier scour. Where footings are placed in non-scourable rock, they shall be keyed into the rock a minimum of 1 foot or as recommended by the Geology Program.

Steel Piling Design

STEEL PILING

The spacing of piling used in pile groups to support footings shall be a minimum of 2’-6” center to center or 2.5 times the pile width, whichever is greater. Piles shall be placed 1’-6” minimum from edge of concrete to centerline piles which includes allowance for pile misalignment. The tops of steel piles shall be embedded 1’-0” minimum into concrete footings and concrete caps of pile bents and abutments.

When the lateral resistance of the soil surrounding the piles is inadequate to counteract the horizontal forces, battered piles may be considered in the substructure. In no case shall any pile be battered greater than 3” in 12” unless approved by Bridge Program Staff.

Steel pile sizes most commonly used are HP 12x53, HP 12x74, and HP 14x73, and HP 14x89. Larger sizes may be used with approval of Bridge Program Staff. In no case shall a pile size less than HP12x53 be used. Grade 50 piling shall be used.

Steel piles, when subjected to uplift, should be provided with adequate anchorage devices such as reinforcing bars or angles welded to the piles. For Service Limit State, pile uplift shall be zero. When verifying geotechnical resistance for uplift resistance, a reduced resistance factor is required. This shall be verified through the Geology Program as this is not explicitly shown in the Geology Report.

The Geology Report will give information relative to foundation materials and other conditions to be encountered in the field in connection with the pile driving. The report will also provide geotechnical design values. It is common in Wyoming to find competent bedrock within a reasonable depth below the substructures. In that case, end bearing piles shall be used. For most hard rock, the Geology Report will specify refusal depth. Where softer soil layers underlie harder soil layers, engineering judgement shall be used to make sure the piles do not puncture through the softer layers. Preboring may be required in these instances.

Where bedrock is not present, the Geology Report will provide friction and end bearing parameters that can be used to estimate pile lengths. Pile Dynamic Analyzer (PDA) testing will be required on friction and end bearing piles to verify pile capacity during construction or as recommended by Geology Program.

The following procedure shall be used to determine the structural capacity of a pile fully supported by soil driven to refusal:

Resistance Factors - Article 6.5.4.2

The following resistance factor will be used for all end bearing piles, including piles not requiring pile tips:

$$\varphi_c = 0.50$$

Resistance / Capacity - Articles 6.9.2.1 and 6.9.4.1

The nominal resistance for piles driven to refusal in bedrock or competent bearing strata will be controlled by the structural limit state. The factored structural resistance in compression, P_r, will be taken as:

$$P_r = \varphi_c P_n$$

For pile bents, use the equations shown in Articles 6.9.2 through 6.9.4. These piles will need to be designed for both moment and axial loads.

In determining the flexural capacity of the strong axis of H-pile members, the cross section limits in Art 6.10.2.2-3 do not apply.

For abutments:

If the pile is fully braced, "l" is taken as zero in equation 6.9.4.1.2-1, and Eq. 6.9.4.1.3-1 is not applicable (See C6.9.4.1.3). Therefore, Pe is infinite, Pe / Po > .44, and Eq. 6.9.4.1.1-1 is applicable, with Po / Pe = 0.

Pn = Po = Fy*Ag.

The pile can be fully braced if it is not subject to scour and can be assured that the soil will adequately provide lateral support. Engineering judgement will be required.

Note: Section 6.9.4.2 need not be checked.

This equation will typically yield a smaller pile size than allowable stress design methodology. Many times a 10" pile may be found to be adequate but should not be used due to drivability concerns.

Where piles are driven to refusal in competent material, pile group capacity does not need to be considered.

Drilled Shaft Foundation Design

Drilled Shaft Foundations may be economical in certain situations, but require specialized equipment and access to the bridge site. Inherently drilled shafts are more difficult to construct than pile foundations and are prone to defects after construction. Cross Hole Sonic (CSL) testing will be considered on most drilled shaft installations to confirm their integrity. The formation in which the holes are to be drilled must be of such a nature that the holes will retain their shape. If caving of the hole is possible, then casing is required. Dewatering is required in stream beds or areas of high ground water. All of these factors shall be considered before selecting drilled shafts.

The Geology Program will recommend casing and dewatering. Permanent casing shall not be allowed unless approved by Bridge Staff.

The Geology Report will give information relative to foundation materials and will also provide geotechnical design values for each strata. The diameter of columns and drilled shaft diameters shall be sized in 6" increments. Where the column diameter and the shaft diameter are equal, the drilled shaft concrete cover shall be provided full length of the column. The minimum diameter of drilled shafts shall be 30 inches. The maximum diameter of drilled shafts shall be 60 inches. Larger size drilled shafts shall be approved by the Bridge Program Staff.

The Geology Program shall be consulted for casing recommendations. Where segmental casing is required, additional concrete cover is required. The shaft diameters for segmental casing are typically in metric sizes.

The following table shall be used for minimum concrete cover for all drilled shafts. This table is adequate for segmental casing and standard casing:

Drilled Shaft Diameter	**Minimum Concrete Cover**
3.0 feet or less	3.0 inches
Greater than 3.0 feet and less than 5.0 feet	4.0 inches
5.0 feet or larger	6.0 inches

In no case shall drilled shaft spacing be less than 4 drilled shaft diameters without approval of Bridge Program Staff. Interaction effects between adjacent shafts will need to be considered when drilled shaft spacing is less than 4 shaft diameters. Sequencing of drilled shaft construction will need to be considered for drilled shaft spacing less than 6 shaft diameters. The requirements for sequencing shafts will be shown on the Substructure Layout sheets.

Drilled Shafts Founded in Rock

For most situations, use either the capacity of the shaft in skin friction or the capacity of the shaft in end bearing alone. Since the mobilization curves for most rock in Wyoming yield little contribution to both skin friction and end bearing, the combined capacity of skin friction and end bearing should only be used when skin friction or end bearing alone results in an uneconomical shaft and the combined capacity is significantly higher than skin friction or end bearing alone. Combined skin friction and end bearing for shafts founded in rock need to consider the movement required to mobilize skin friction versus the movement that is required to mobilize end bearing. Using skin friction is preferred over end bearing due to uncertainty in bearing layers of subsurface material. Before considering skin friction in rock, the Geology Program shall be consulted to confirm that the material's side walls are not prone to degradation over time.

Typically, strength limit state will control for drilled shafts founded in rock. For normal situations service limit state will not typically control when using 100% skin friction or 100% end bearing for strength limits state geotechnical resistance. When checking the Service limit state, nominal resistance of the rock should be used in accordance with the LRFD specification.

Drilled Shafts in Soil or IGM

For shafts in soils or IGMs, without the presence of rock, end bearing and skin friction may be directly combined for the strength limit state. In weak interbedded soils, designing for skin friction only should be considered.

For service limit state, consideration shall be given to mobilization of the shaft. When checking the Service limit state, nominal resistance of the soil or IGM should be used in accordance with the LRFD specification.

Where the area of spiral reinforcement is not controlled by seismic requirements, shear or torsion, spiral reinforcement requirement in Article 5.6.4.6 can be ignored for drilled shafts.

The minimum transverse reinforcement in drilled shafts shall be a No. 5 spiral with a 6” pitch, unless additional reinforcement is required by design (i.e., for seismic or vehicle collision load).

Column Design

General Column Design

Columns shall be sized to limit their slenderness ratio (kl/r) to less than 100 unless approved by Bridge Staff. *Approximate Evaluation of Slenderness Effects* in accordance with the LRFD Bridge Design Specifications shall be used. BRASS Pier shall be used to evaluate concrete columns. Second order affects for steel piles shall be considered using simplified analysis such as moment magnification to account for secondary affects . If this analysis requires the use of a larger steel pile, a more refined analysis should be used such as P-Delta.

Drilled shafts shall be considered concrete columns for structural resistance. Column interaction shall also be considered.

When the diameter of the drilled shaft is no more than 6” larger than the column diameter, the drilled shaft shall be assumed the same size as the column for analysis purposes.

The design column length shall be simplified and based on a point of “fixity”. One point of fixity shall be used for all load cases using the equivalent cantilever method using L-pile or All Pile software.

The following methodology provides a good starting point for determining point of fixity:

I. Determine the general classification of soil along the length of the shaft from the Log Boring Sheet.

II. If crossing a waterway, determine the magnitude of design scour due to the design flood from the

Hydraulic Scour Report.

III. Refer to National Center for Highway Research Program (NCHRP) 343 entitled "Manuals for The Design of Bridge Foundations," dated December 1991. Section 4.1 of this text provides a discussion of the Davisson and Robinson Method for determining equivalent free standing length of drilled shaft foundations. A summary of the method and equations used follows.

A. For soils composed primarily of clays:

1. Calculate the soil modulus (E_S) based on the following.

$E_s = 67S_u$ in units of force/length

Where S_u = Undrained shear strength in units of force/length^2

2. Calculate R based on the following.

$R = [(E_pI_p)/E_s]^{0.25}$ (units of length)

Where E_p = Young's modulus of drilled shaft in units of force/length^2

I_p = Moment of inertia of drilled shaft in units of length^4

E_s = Soil modulus in units of force/length^2

3. Calculate the equivalent length (L_{eq}).

$L_{eq} = L_u + 1.4R$ (units of length)

Where L_u = Unsupported length of shaft extending above ground in units of length

B. For soils composed primarily of sands:

1. Estimate the rate of increase in soil modulus with depth (n_h) from the following table.

RECOMMENDED VALUES OF n_h, (lb/in^3)		
Density	Dry or Moist Sand	Submerged Sand
Loose	30	15
Medium	80	40
Dense	200	100

2. Calculate T based on the following.

$T = [(E_pI_p)/N_h]^{0.2}$ (units of length)

Where E_p = Young's modulus of drilled shaft in units of force/length^2
I_p = Moment of inertia of drilled shaft in units of length^4

3. Calculate the equivalent length (L_{eq}).

$L_{eq} = L_u + 1.8T$
Where L_u = Unsupported length of shaft extending above ground in units of length

IV.

To ensure column stability, check minimum embedment (L_{min}) of shaft below ground line, or below design scour elevation for bridges crossing a waterway. This procedure, which uses the same parameters as previously defined, is described by Swan and Wright in Chapter 3 of the "Center for Transportation Research Report #415-1," dated November 1986.

A. For soils composed primarily of clays:

$$L_{min} = (4.8)(4^{0.25})(R) = (6.7882)(R)$$

B. For soils composed primarily of sands:

$$L_{min} = 4.8T$$

The minimum depth of the column below point of fixity will be determined by the L-Pile or All Pile analysis. The depth shall be as necessary to have a stable column (i.e. reverse curvature of deflection curve). In weak soils, the minimum depth of drilled shaft below the theoretical point of fixity shall be 10 feet.

Often times when a drilled shaft is socketed into rock, the point of fixity will be close to the rock surface. The minimum embedment of drilled shafts into rock shall be 5 feet.